11 DESIGN OF COLD-FORMED STEEL OPEN SECTIONS, SHEET PROFILES, HOLLOW SECTIONS AND SHEET PILE SECTIONS

This section gives recommendations for the design of cold-formed steel sections.

Clauses 11.1 to 11.6 give recommendations for the design of cold-formed thin gauge steel open sections and sheet profiles with nominal thickness up to 4 mm (This nominal thickness includes the coating, which only applies to thin gauge steel open sections and sheet profiles). Tensile strength and ductility requirements shall comply with clause 11.2.2. The use of these sections and sheet profiles should be either justified by design calculation or by testing. These thin gauge steel open sections and sheet profiles are normally manufactured by cold-rolled forming process. For other manufacturing process such as press-braking process or bend-braking process, the curving or straightening requirements as stipulated in clause 14.2.7 should be complied with.

Clause 11.7 gives recommendations for the design of cold-formed steel hollow sections with nominal thickness up to 22 mm. Clause 11.8 gives recommendations for the design of cold-formed steel sheet pile sections with nominal thickness up to 16 mm. For cold-formed steel open sections and sheet profiles with nominal thickness greater than 4 mm, tensile strength and ductility requirements shall comply with clause 3.1.2.

11.1 GENERAL DESIGN OF OPEN SECTIONS AND SHEET PROFILES

Clauses 11.1 to 11.6 give recommendations for the design of cold-formed thin gauge steel open sections and sheet profiles with nominal thickness up to 4 mm as shown in Figure 11.1.

Design recommendations for thin gauge steel open sections and sheet profiles with the following applications are provided:

Open sections:	Secondary structural elements such as purlins and side rails, and primary structural members in trusses and portal frames of modest span.
Sheet profiles:	Floor decking as well as roof and wall cladding.

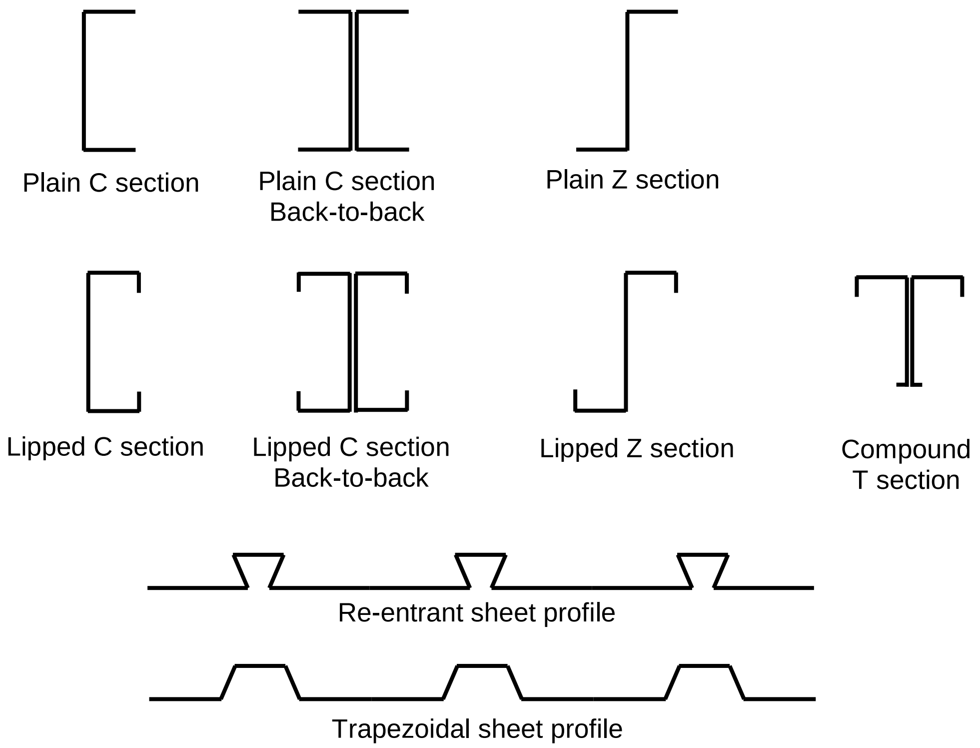


Figure 11.1 - Typical cold-formed steel open sections and sheet profiles

The yield strength of the cold-formed thin gauge steel open sections and sheet profiles shall not be greater than 550 N/mm^2. For welding of cold-formed steel, refer to Clause 11.7.5 and Table 11.5.

No specific design rules for cold-formed thin gauge steel open sections and sheet profiles with intermediate stiffeners are given this clause. The use of these sections and sheet profiles should be either justified by other established design procedures or by testing.

11.2 MATERIAL PROPERTIES

11.2.1 Physical properties

The physical properties of the steel strips are given in clause 3.1.6.

The design thickness of the material shall be taken as the nominal base metal thickness exclusive of coatings.

11.2.2 Mechanical properties

Both the yield and the tensile strengths, and hence the ductility, of the steel strips with nominal thickness up to 4 mm shall comply with clause 3.8.1.

11.2.2.1 Effects of cold forming

The increase in yield strength due to cold working shall not be utilized for members which undergo welding, annealing, galvanizing or any other heat treatment after forming which may produce weakening.

The increase in yield strength due to cold forming shall be taken into account by replacing the material yield strength, Y_s, by the average yield strength, Y_{sa}, of the sections or the sheet profiles.

For elements under tension, the full effect of cold working on the yield strength shall be used, and thus the design strength, p_y, shall be taken as Y_{sa}.

$$p_y = Y_{sa} \tag{11.1}$$

For elements of flat width, b, and thickness, t, under compression, the design strength, p_y, shall be taken as the average yield strength in compression, Y_{sac} which is given by:

$$p_y = Y_{sac} \tag{11.2}$$

For stiffened elements:

$$Y_{sac} = Y_{sa} \quad \text{for} \quad b/t \leq 24\,\varepsilon \tag{11.3a}$$

$$= Y_s \quad \text{for} \quad b/t \geq 48\,\varepsilon \tag{11.3b}$$

For unstiffened elements:

$$Y_{sac} = Y_{sa} \quad \text{for} \quad b/t \leq 8\,\varepsilon \tag{11.3c}$$

$$= Y_s \quad \text{for} \quad b/t \geq 16\,\varepsilon \tag{11.3d}$$

$$\text{where} \quad \varepsilon = \sqrt{\frac{275}{Y_s}} \tag{11.4}$$

For intermediate values of b/t, the value of Y_{sac} shall be obtained by linear interpolation.

The average yield strength, Y_{sa} is given by:

$$Y_{sa} = Y_s + \frac{5Nt^2}{A}(U_s - Y_s) \leq 1.25\,Y_s \quad \text{or} \quad \leq U_s \tag{11.5}$$

where

N is the number of full 90° bends in the section with an internal radius < 5t (fractions of 90° bends shall be counted as fractions of N);

t is the net thickness of the steel strip (mm);

U_s is the ultimate tensile strength (N/mm^2);

A is the gross area of the cross-section (mm^2).

Alternatively, the value of Y_{sa} shall be determined by tests.

11.3 SECTION PROPERTIES

11.3.1 Gross section properties

Section properties shall be calculated according to good practice, taking into account of the sensitivity of the properties of the gross cross-section to any approximations used and their influence on the predicted section capacities and member resistances.

When calculating the section properties of sections up to 3.2 mm thickness, the steel material is assumed to be concentrated at the mid-line of the strip thickness, and the actual round corners are replaced by sharp corners, i.e. intersections of the flat elements, as shown in Figure 11.2a.

When calculating the section properties of sheet profiles up to 2.0 mm thickness, the steel material is assumed to be concentrated at the mid-line of the strip thickness, as shown in Figure 11.2b, provided that the flat width of all the elements is greater than 6.7 r where r is the internal corner radius, or 20 t, whichever is the greatest.

Moreover, the presence of corners and bends shall be allowed for as shown in Table 11.1:

Table 11.1 - Basis for calculation of section properties

Geometrical limits	Basis for calculation
$r \leq 5t$	Round corners to be replaced with sharp corners, i.e. intersects of flat elements
$5t < r \leq r_o$	Round corners should be used.
$r > r_o$	Section capacities should be determined by testing.

where r_o is the limiting radius of effective corners

$$= 0.04\ t\ E / p_y \qquad (11.6)$$

Hence, the effective width of a flat element, b, shall be generally calculated on the assumption that each element extends to the mid-points of the corners. Refer to clause 11.3.4.4.2 for elements with corners in which r is larger than 5 t.

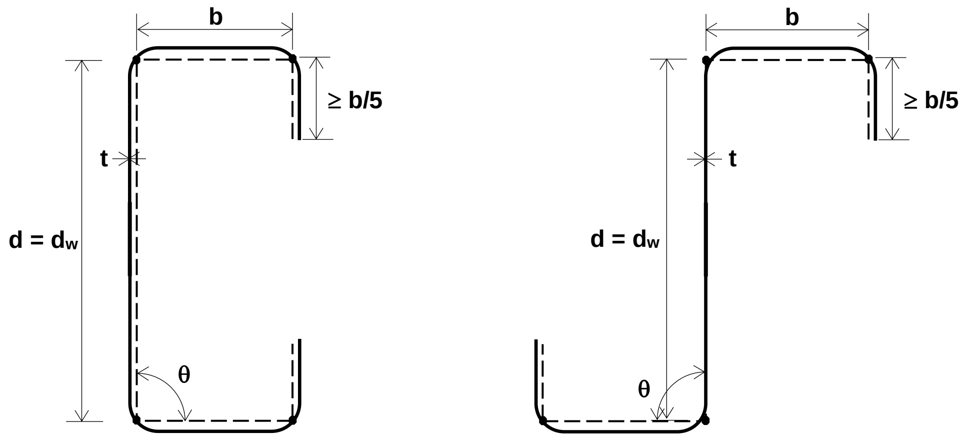


Figure 11.2a - Idealized section for C and Z section

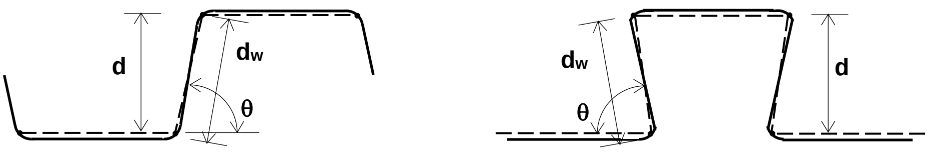


Figure 11.2b - Idealized section for sheet profile

In general, holes for fasteners are not necessary to be deducted when calculating cross-section properties, but allowance shall be made for large openings or arrays of small holes.

The net section properties of sections with regular or irregular arrays of holes, other than holes required for fastening and filled with bolts, shall be determined by established design procedures or by testing.

11.3.2 Effective section properties under tension

In general, the gross section defined by the mid-line dimensions in clause 11.3.1 shall be adopted.

The effect of bolt holes shall be taken into account in determination of the design strength and stiffness of sections and sheet profiles under tension, and the net area, A_n, shall be taken as the gross area less deductions for holes and openings.

In general, when deducting for holes for fasteners, the nominal hole diameter shall be used. However, for countersunk holes, the area to be deducted shall be the gross cross-sectional area of the hole, including the countersunk portion, in the plane of its axis.

11.3.3 Effective section properties under compression and bending

In general, the gross section defined by the mid-line dimensions in clause 11.3.1 shall be adopted and modified accordingly.

The effects of local buckling shall be taken into account in determination of the design strength and stiffness of sections and sheet profiles under compression and bending. This shall be accomplished by using effective cross-sectional properties which are calculated on the basis of the effective widths of those elements that are prone to local buckling or by the effective stress method.

The effects of intermediate stiffeners and bends shall also be incorporated in determining the effective section properties, as appropriate.

In determining effective section properties for strength assessment, the maximum stresses in flat elements prone to local buckling shall be determined according to the design loads at ultimate limit state, and not exceeding the design strength of the steel material. However, in determining effective section properties for stiffness assessment, the maximum stresses in flat elements prone to local buckling shall be determined according to the design loads at serviceability limit state.

Effective section properties shall be evaluated in accordance with established design procedures. Moreover, the corresponding maximum width-to-thickness ratios shall be observed.

The effect of distortional buckling in sections shall be incorporated, especially in sections and steel profiles with high strength steels. However, it should be noted that for sections and steel profiles with design strengths at 350 N/mm^2 or below, distortional buckling in sections and steel profiles within the practical ranges of dimensions shall be neglected.

The possible shift of the centroidal axis of the effective cross-section relative to the centroidal axis of the gross cross-section shall be taken into account.

11.3.4 Local buckling

11.3.4.1 General

A rigorous non-linear finite element analysis could be used in determining the resistance of a general cross section. Alternatively, the effects of local buckling in strength and stiffness assessments in sections and sheet profiles shall be allowed for through the use of effective cross sections which comprise of

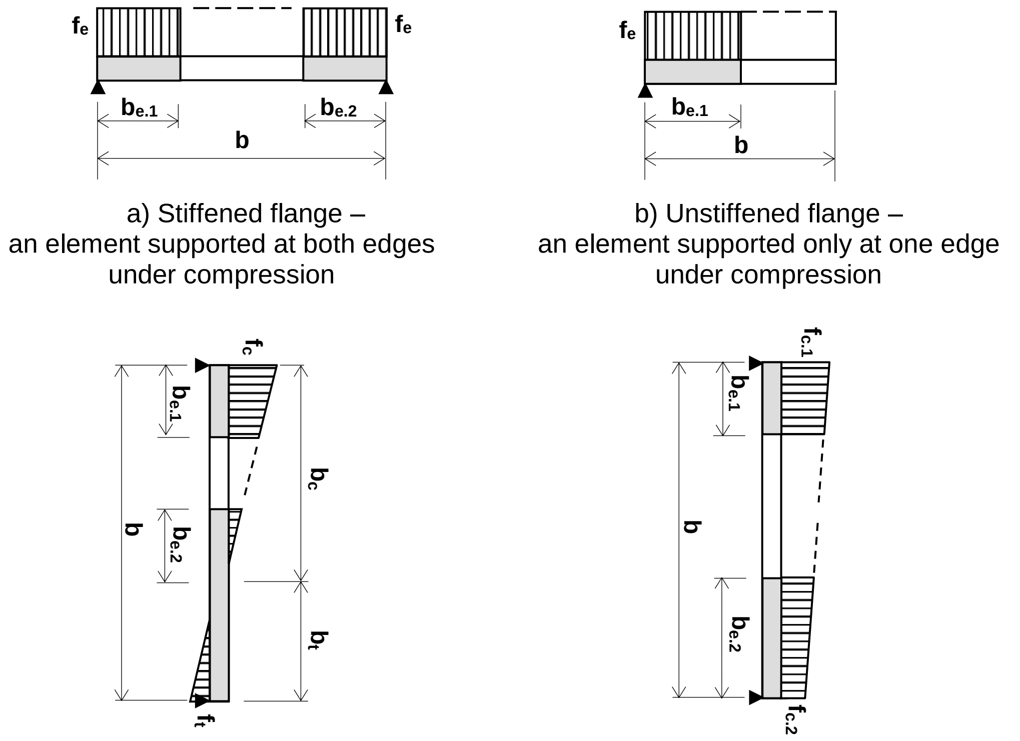
a) the effective areas of individual flat elements wholly or partly under compression,

b) the effective areas of intermediate stiffeners, and

c) the full areas of individual flat elements under tension.

For a flat stiffened element, the effective area consists of two portions as shown in Figure 11.3a), i.e. one adjacent to each supported edge. Refer to clause 11.3.4.4.3 for details.

For a flat unstiffened element, the whole of the effective area is located adjacent to the supported edge as shown in Figure 11.3b). Refer to clause 11.3.4.4.4 for details.



a) Stiffened flange –
an element supported at both edges
under compression

b) Unstiffened flange –
an element supported only at one edge
under compression

c) Stiffened web –
an element supported at both edges
under bending

d) Stiffened web –
an element supported at both edges
under combined compression and bending

Figure 11.3 - Effective width for stiffened and unstiffened elements

11.3.4.2 *Maximum width to thickness ratios*

For flat elements under compression, the maximum values of element flat width to thickness ratio, $(b / t)_{max}$, covered by the design procedures given in this clause are as follows:

(a) Stiffened elements with one edge connected to a flange element or a web element and the other edge supported with:	(i) a simple lip	60 ε
	(ii) any other type of stiffener with sufficient flexural rigidity	90 ε
(b) Unstiffened elements		60 ε
(c) Stiffened elements with both edges connected to other stiffened elements		500 ε

where

ε is $\sqrt{\frac{275}{p_y}}$;

It should be noted that unstiffened compression elements that have width to thickness ratios b/t exceeding 30ε and stiffened compression elements that have *b / t* ratios exceeding 250ε are likely to develop noticeable deformations at the full working load, without affecting the ability of the member to carry this load.

11.3.4.3 *Stiffened elements under edge stiffener*

For a flat element to be considered as a stiffened element under compression, it should be supported along one edge by a flange or a web element, while the other edge supported by (i) a web, (ii) a lip or (iii) other edge stiffener which has adequate flexural rigidity to maintain the straightness of this edge under load.

Irrespective of its shape, the second moment of area of an edge stiffener about an axis through the mid-thickness of the element to be stiffened shall not be less than I_{min} where I_{min} is given by:

$$I_{\min} = \frac{t\, b^3}{375} \tag{11.7}$$

where

b is the width of the element to be stiffened;

t is the thickness.

Where a flat element is stiffened by a simple lip, the lip shall be at an angle of not less than 70° from the element to be stiffened.

Where the stiffener consists of a simple lip at right angles to the element to be stiffened, a width of lip not less than one-fifth of the element width b, as indicated in Figure 11.4, shall be taken as satisfying this condition.

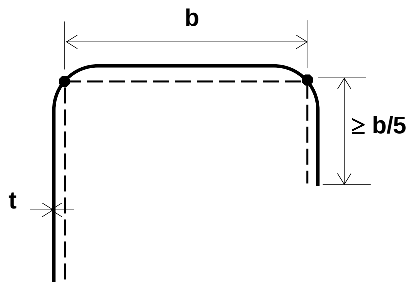


Figure 11.4 - Simple lip edge stiffener

11.3.4.4 Effective width for strength calculation

11.3.4.4.1 Basic effective width

The effective width, b_e , of a flat element under uniform compression with a flat width b is given by:

$$b_e = \beta\, b \tag{11.8}$$

where

$$\beta = 1.0 \quad \text{when } \rho \le 0.123 \tag{11.9a}$$

$$= \left\{1 + 14(\sqrt{\rho} - 0.35)^4\right\}^{-0.2} \quad \text{when } \rho > 0.123 \tag{11.9b}$$

where

$$\rho = \frac{f_c}{p_{cr}} \tag{11.10}$$

f_c is the applied compressive stress in the effective element; $\le p_y$

p_{cr} is the local buckling strength of the element.

$$= 0.904\, E\, K\left(\frac{t}{b}\right)^2 \tag{11.11}$$

where

K is the relevant local buckling coefficient;

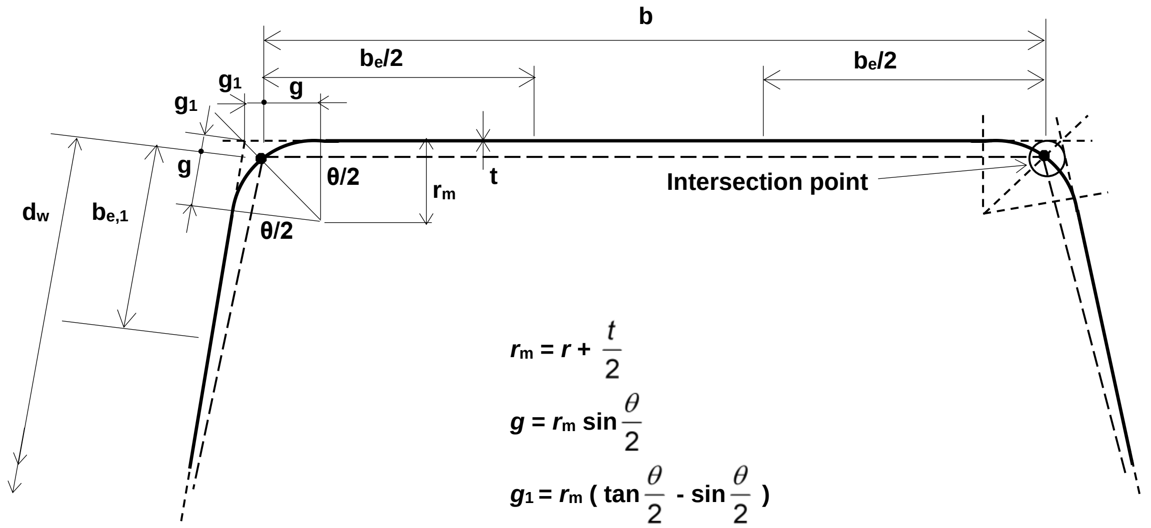
t is the net thickness of the steel material;

b is the flat width of the element.

The local buckling coefficient K depends upon the type of element and also the geometry of the sections and the sheet profiles; refer to clauses 11.3.4.4.3 and 11.3.4.4.4 for details.

11.3.4.4.2 Effect of large radius

When the internal radius r of a corner exceeds $5t$, the effective width of each of the flat elements meeting at that corner should be reduced by $r_m \sin(\theta/2)$, as shown in Figure 11.5.



where

r is the internal bend radius;
r_m is the mean bend radius;
t is the steel thickness;
θ is the angle between the web and the flange;
g, g_1 are connections to element lengths at corner radii.

Figure 11.5 - Calculation of effective widths allowing for corner radii

11.3.4.4.3 Effective width of a flat stiffened flange element

The effective width of a flat stiffened element forming a compression flange of a section or a sheet profile shall be determined in accordance with clause 11.3.4.4.1, using the appropriate value of K.

For flanges stiffened at both longitudinal edges under uniform compression, the value of the buckling coefficient K shall conservatively be taken as 4. Alternatively, a more precise value of K shall be obtained as follows:

$$K = 5.4 - \frac{1.4\,h}{0.6 + h} - 0.02\,h^3 \quad \text{for sections} \quad (11.12a)$$

$$= 7 - \frac{1.8h}{0.15 + h} - 0.091h^3 \quad \text{for sheet profiles} \quad (11.12b)$$

where

h = d_w / b;
d_w is the sloping distance between the intersection points of a web and the two flanges;
b is the flat width of the flange.

11.3.4.4.4 Effective width of a flat unstiffened flange element

The effective width b_{eu} of a flat unstiffened element under uniform compression is given by

$$b_{eu} = 0.89\, b_e + 0.11b \quad (11.13)$$

where

b_e is determined from the basic effective width determined in accordance with clause 11.3.4.4.1;
b is the flat width of the element.

The value of K shall conservatively be taken as 0.425 for any unstiffened element. Alternatively a more precise value of K shall be obtained as follows:

$$K = 1.28 - \frac{0.8h}{2 + h} - 0.0025h^2 \quad (11.14)$$

11.3.4.4.5 Effective width of a flat web element

The web shall be considered to be fully effective when

i) the web depth to thickness ratio $d_w / t \leq 70\,\varepsilon$, or

ii) both edges of the web are under tension.

In all other cases, the effective width of a web in which the stress varies linearly as shown in Figure 11.6, should be obtained in two portions, one adjacent to each edge as follows:

a) One edge in tension (see Figure 11.6a):

$$b_{e,1} = 0.76t\sqrt{\frac{E}{f_{c,1}}} \qquad (11.15a)$$

$$b_{e,3} = 1.5\,b_{e,1} \qquad (11.15b)$$

where

$b_{e,1}$ is the portion of the effective width adjacent to the more compressed edge;

$b_{e,3}$ is the portion of the effective width adjacent to the tension edge;

$f_{c,1}$ is the larger compressive edge stress;

b_t is the portion of the web under tension;

E is the modulus of elasticity;

t is the net thickness of the steel material.

If $b_{e,1} + b_{e,3} + b_t \geq d_w$, then the web is fully effective, where d_w is the sloping distance between the intersection points of a web and the two flanges.

b) Both edges in compression (see Figure 11.6b):

$$b_{e,1} = 0.76t\sqrt{\frac{E}{f_{c,1}}} \qquad (11.16a)$$

$$b_{e,2} = \left(1.5 - 0.5\frac{f_{c,2}}{f_{c,1}}\right)b_{e,1} \qquad (11.16b)$$

where

$b_{e,1}$ is the portion of the effective width adjacent to the more compressed edge;

$b_{e,2}$ is the portion of the effective width adjacent to the less compressed edge;

$f_{c,1}$ is the larger compressive edge stress;

$f_{c,2}$ is the smaller compressive edge stress;

E is the modulus of elasticity;

t is the net thickness of the steel material.

In both cases, if $b_{e,1} + b_{e,2} \geq d_w$, then the web is fully effective, where d_w is the sloping distance between the intersection points of a web and the two flanges.

If the location of the neutral axis is determined iteratively using effective section properties rather than assuming the web to be fully effective, then $b_{e,1}$ in items a) and b) above should be determined as follows:

$$b_{e,1} = 0.95t\sqrt{\frac{E}{f_{c,1}}} \qquad (11.17)$$

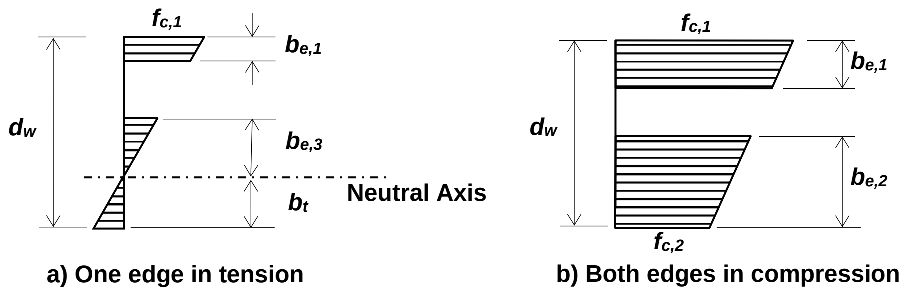


Figure 11.6 - Stress distributions over effective portions of web

11.3.4.5 Effective width for deflection calculation

11.3.4.5.1 Flat flange elements

When calculating deflection, the effective width $b_{e,ser}$ for a stiffened or an unstiffened flat flange element is given by:

a) when $\lambda_{ser} \le \lambda_1$:
$$b_{e,ser} = \frac{1.27\,b}{\lambda_{ser}^{2/3}} \text{ but } b_{e,ser} \le b \tag{11.18a}$$

b) when $\lambda_1 < \lambda_{ser} \le \lambda$:
$$b_{e,ser} = b_{e,1,ser} + (b_e - b_{e,1,ser})\frac{(\lambda_{ser} - \lambda_1)}{(\lambda - \lambda_1)} \tag{11.18b}$$

where

$$b_{e,1,ser} = \frac{1.27\,b}{\lambda_1^{2/3}} \text{ but } b_{e,1,ser} \le b; \tag{11.19}$$

$$\lambda = \frac{2\,b/t}{\sqrt{K}}\sqrt{\frac{p_y}{E}}; \tag{11.20}$$

$$\lambda_1 = 0.51 + 0.6\lambda; \tag{11.21}$$

$$\lambda_{ser} = \frac{2\,b/t}{\sqrt{K}}\sqrt{\frac{f_{ser}}{E}}; \tag{11.22}$$

b_e is the basic effective width determined in accordance with clause 11.3.4.4.1;
f_{ser} is the compressive stress in the effective element at serviceability limit state;
K is the relevant local buckling coefficient determined in accordance with clause 11.3.4.4.3 or 11.3.4.4.4.

11.3.4.5.2 Flat web elements

When calculating deflections, the web shall be considered to be fully effective when

i) the web depth to thickness ratio $d_w / t \le 150\varepsilon$, or
ii) both edges of the web are under tension.

Where this limit is exceeded, the effective width of a web in which the stress varies linearly as shown in Figure 11.6, should be obtained in two portions, one adjacent to each edge as follows:

a) One edge in tension (see Figure 11.6a):

$$b_{e,1,ser} = 0.95\,t\sqrt{\frac{E}{f_{c,1,ser}}} \tag{11.23a}$$

$$b_{e,3,ser} = 1.5\,b_{e,1,ser} \tag{11.23b}$$

where

$b_{e,1,ser}$ is the portion of the effective width adjacent to the more compressed edge;
$b_{e,3,ser}$ is the portion of the effective width adjacent to the tension edge;
$b_{t,ser}$ is the portion of the web under tension at serviceability limit state.

If $b_{e,1,ser} + b_{e,3,ser} + b_t \ge d_w$ where d_w is the sloping distance between the intersection points of a web and the two flanges, then the web is fully effective at serviceability limit state.

b) Both edges in compression (see Figure 11.6b):

$$b_{e,1,ser} = 0.95\,t\sqrt{\frac{E}{f_{c,1,ser}}} \tag{11.24a}$$

$$b_{e,2,ser} = (1.5 - 0.5\frac{f_{c,2,ser}}{f_{c,1,ser}})b_{e,1,ser} \tag{11.24b}$$

where

$f_{c,1,ser}$ is the larger compressive edge at serviceability limit state;
$f_{c,2,ser}$ is the smaller compressive edge at serviceability limit state;
$b_{e,1,ser}$ is the portion of the effective width adjacent to the more compressed edge;

$b_{e,2,ser}$ is the portion of the effective width adjacent to the less compressed edge.

If $b_{e,1,ser} + b_{e,2,ser} \geq d_w$ where d_w is the sloping distance between the intersection points of a web and the two flanges, then the web is fully effective at serviceability limit state.

11.3.5 Flange curling

Sections and sheet profiles with flanges which have high width to thickness ratios b / *t* are susceptible to exhibit the type of cross-sectional distortion known as 'flange curling' shown in Figure 11.7. Provided that *b* / *t* is not greater than 250 ε for sections and 500 ε for sheet profiles, the inward movement of each flange towards the neutral axis shall be assumed to be less than 0.05 D_p, where D_p is the overall depth of the section or the sheet profile, and its occurrence shall be neglected for structural purposes.

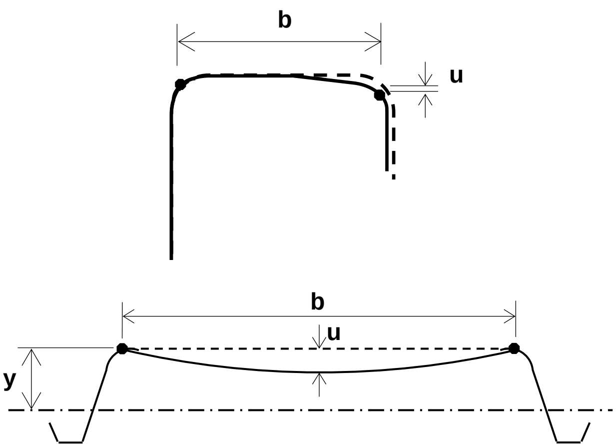


Figure 11.7 - Flange curling

When necessary, the maximum inward movement *u* of the flange towards the neutral axis should be determined as follows:

$$u = 2\frac{f_a^{\,2}\, b_{fc}^{\,4}}{E^2\, t^2\, y} \tag{11.25}$$

where

f_a is the average stress in the flange;

b_{fc} is the width of the flange for flange curling as shown in Figure 11.7;

= the flange width, *b*, for an unstiffened or edge stiffened flange in sections, or

= half of the flange width, i.e. 0.5 *b* , for a stiffened flange in sheet profiles;

E is the modulus of elasticity;

t is the net thickness of steel material;

y is the distance from the flange to the neutral axis.

This equation is applicable to both compression and tension flanges with or without stiffeners.

NOTE: If the stress in the flange has been calculated on the basis of an effective width, b_e, then f_a can be obtained by multiplying the stress on the effective width by the ratio of the effective flange area to the gross flange area.

11.4 MEMBERS UNDER LATERAL LOADS

11.4.1 General

This clause is concerned with sections and sheet profiles which are subjected to lateral loads, and section capacities against bending, shear and crushing acting separately and in combination.

In general, the moment capacities shall be determined using the non-linear finite element analysis allowing for imperfections and second-order effects could be used in place of the following effective length method or the effective cross sections incorporating the effective areas of those elements partly or wholly in compression and the effective areas of all stiffeners as well as the gross areas of those elements under tension. The moment capacities shall be based on the attainment of a limiting compressive stress equal to the design strength, p_y , in the effective cross sections. For sections or sheet profiles where the webs are only partly effective, iterations shall be performed to locate the actual positions of the neutral axes of the effective cross sections for improved performance.

In cases where the tensile stress reaches the design strength p_y before the compressive stress, plastic redistribution of tensile stresses due to tension yielding shall be taken into account for enhanced capacities.

When calculating deflections, the effective section areas of those elements partly or wholly in compression should be determined under serviceability loads.

11.4.2 Moment capacity

11.4.2.1 Laterally stable beams

This clause is concerned with sections and sheet profiles which are laterally stable. Lateral torsional buckling of sections and sheet profiles shall be checked in accordance with clause 11.4.7.

11.4.2.2 Determination of moment capacity

The effective cross section of a section or a sheet profile comprising flat flange and web elements, as indicated in Figure 11.8, shall be determined as follows.

a) *Effective section with local buckling in compression flange*
The effective section comprises of
- the effective area of the compression flange;
- the gross area of the web; and
- the gross area of the tension flange.

The compression flange shall be supported at both edges, and the effective area shall be determined under a compressive stress f_c equal to the design strength p_y.

b) *Effective section with local bucking in both compression flange and web*
The effective section comprises of
- the effective area of the compression flange;
- the effective area of the web; and
- the gross area of the tension flange.

The effective area of the web shall be determined under a compressive stress f_c equal to the design strength p_y at the flange - web junction. It should be noted that iterations are usually required to locate the actual position of the neutral axis.

The moment capacity M_c is given by:

$$M_c = p_y I_e / y_c \quad \text{if} \quad y_c \geq y_t \tag{11.26a}$$

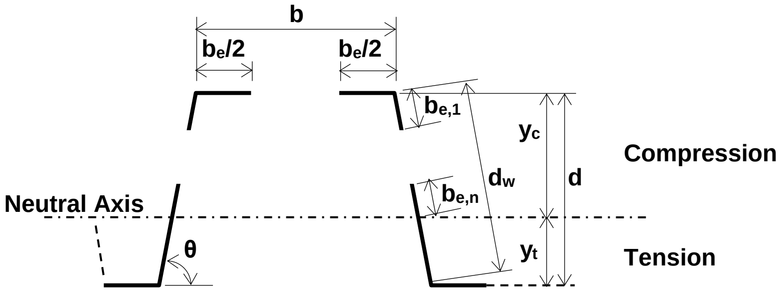
$$= p_y I_e / y_t \quad \text{if} \quad y_t \geq y_c \tag{11.26b}$$

where

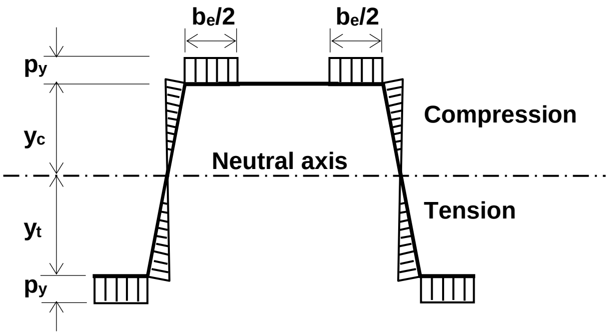
I_e is the second moment of area of the effective cross section which is established in accordance with a) or b) above;

y_c and y_t are as shown on Figure 11.8.

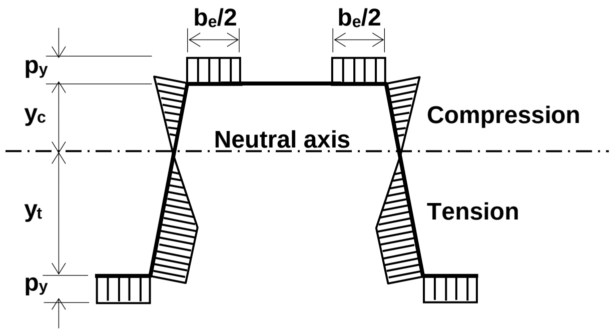
Tension yielding in the web portion under tension, and hence, plastic redistribution of tensile stresses shall be allowed whenever appropriate for increased moment capacities.



a) Effective cross section of an unstiffened trapezoidal profile in bending



b) Elastic stress distribution: b_e based on p_y



c) Partial plasticity in tension zone: b_e based on p_y

Figure 11.8 - Determination of moment capacity

11.4.3 Shear capacity

11.4.3.1 Sections

The maximum elastic shear stress shall not be greater than $0.7p_y$, where p_y is the design strength.

The average shear strength p_v is given by the lesser of the plastic shear strength, $p_{v,y}$ or the shear buckling strength, $p_{v,cr}$, obtained as follows:

$$p_{v,y} = 0.6\ p_y \quad \text{N/mm}^2 \qquad (11.27)$$

$$p_{v,cr} = \left(\frac{1000\ t}{d_w}\right)^2 \quad \text{N/mm}^2 \qquad (11.28)$$

where

p_y is the design strength in N/mm^2;

t is the net thickness of the steel material in mm;

d_w is the sloping distance between the intersection points of a web and the two flanges in mm;

d is the overall depth of the section and sheet profile in mm.

The shear capacity of the web in sections, V_c, is given by

$$V_c = p_{v,y}\, d\, t \quad \text{but} \quad < p_{v,cr}\, d\, t \qquad (11.29)$$

11.4.3.2 Sheet profiles

The average shear strength p_v is given by

$$p_v = 0.6\, p_y \quad \text{if} \quad \lambda_w \le 2.33 \tag{11.30a}$$

$$= 1.4\, \frac{p_y}{\lambda_w} \quad \text{if} \quad 2.33 < \lambda_w \le 4.0 \tag{11.30b}$$

$$= 5.6\, \frac{p_y}{\lambda_w^{\,2}} \quad \text{if} \quad \lambda_w > 4.0 \tag{11.30c}$$

where

λ_w is the web slenderness

$$= \frac{d_w}{t}\sqrt{p_y / E} \tag{11.31}$$

The average shear capacity of the web in sheet profiles is given by

$$V_c = p_v\, d\, t \tag{11.32}$$

where

p_v is the shear strength;
t is the net thickness of the steel material;
d_w is the sloping distance between the intersection points of a web and the two flanges;
d is the overall depth of the sheet profile.

11.4.4 Combined bending and shear

For flat webs of sections and sheet profiles subjected to combined bending and shear action, the following equation should be satisfied:

a) $$\frac{V}{V_c} \le 1 \tag{11.33}$$

b) $$\frac{M}{M_c} \le 1 \tag{11.34}$$

c) $$\left(\frac{V}{V_c}\right)^2 + \left(\frac{M}{M_c}\right)^2 \le 1 \tag{11.35}$$

where

V is the applied shear force;
V_c is the shear capacity;
M is the corresponding applied moment acting at the same cross section as V;
M_c is the moment capacity.

11.4.5 Web crushing capacity

11.4.5.1 Sections

The web crushing capacity of flat section webs, P_w under concentrated forces, either loads or reactions, as shown in Figure 11.9, shall be evaluated using the equations given in Table 11.2a) and Table 11.2b) provided that $45 \le d / t \le 200$ and $r / t \le 6$.

Table 11.2a - Shapes having single thickness webs

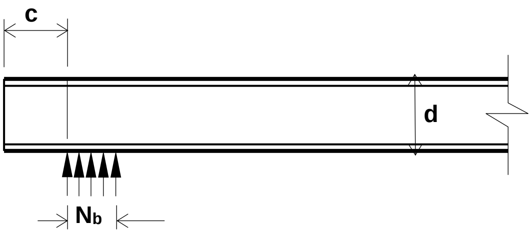
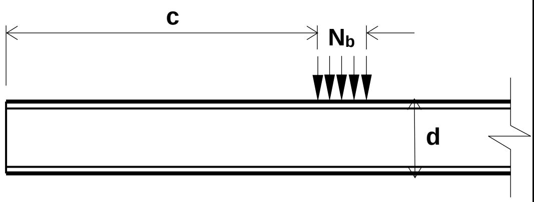
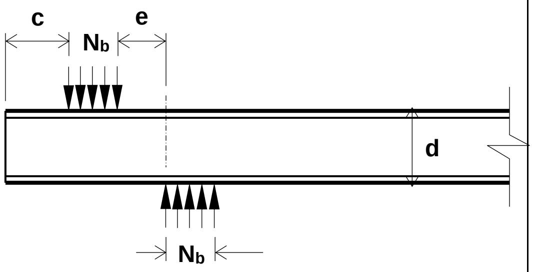
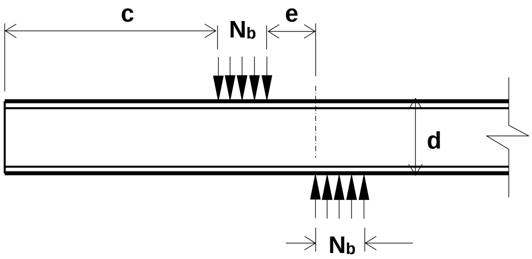
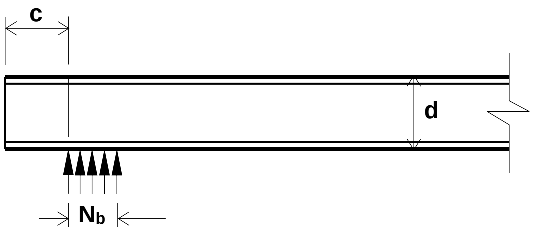
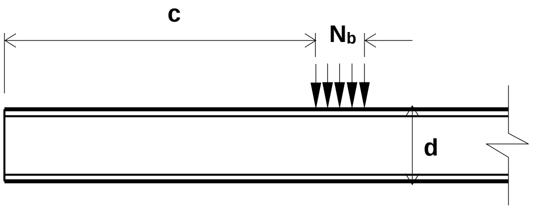
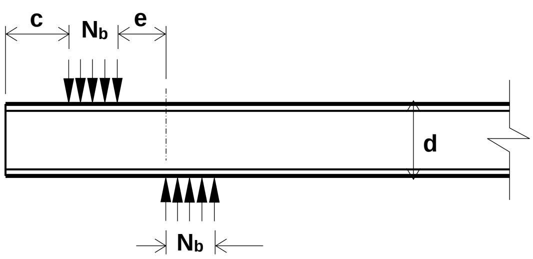
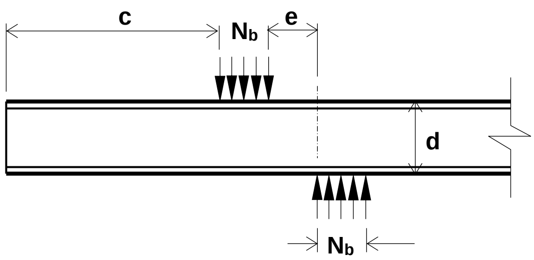
Type and position of loadings	Total web resistance, P_w
i) Single load or reaction  c < 1.5d. Load or reaction near or at free end	Stiffened flanges $P_w = 1.21t^2k_wc_3c_4c_{12}\left\{2060-3.8\frac{d}{t}\right\}\times\left\{1+0.01\frac{N_b}{t}\right\}$ Unstiffened flanges [a] $P_w = 1.21t^2k_wc_3c_4c_{12}\left\{1350-1.73\frac{d}{t}\right\}\times\left\{1+0.01\frac{N_b}{t}\right\}$
ii) Single load or reaction  c < 1.5d. Load or reaction far from free end	Stiffened and unstiffened flanges [b] $P_w = 1.21t^2k_wc_1c_2c_{12}\left\{3350-4.6\frac{d}{t}\right\}\times\left\{1+0.007\frac{N_b}{t}\right\}$
iii) Two opposite loads or reactions e < 1.5d  c ≤ 1.5d. Loads or reactions near or at free end	Stiffened and unstiffened flanges $P_w = 1.21t^2k_wc_3c_4c_{12}\left\{1520-3.57\frac{d}{t}\right\}\times\left\{1+0.01\frac{N_b}{t}\right\}$
iv) Two opposite loads or reactions e < 1.5d  c > 1.5d. Loads or reactions far from free end	Stiffened and unstiffened flanges $P_w = 1.21t^2k_wc_1c_2c_{12}\left\{4800-14\frac{d}{t}\right\}\times\left\{1+0.0013\frac{N_b}{t}\right\}$
[a] When $\frac{N_b}{t}$ > 60, the factor {1+0.01($\frac{N_b}{t}$)} shall be increased to {0.71+0.015($\frac{N_b}{t}$)}	
[b] When $\frac{N_b}{t}$ > 60, the factor {1+0.007($\frac{N_b}{t}$)} shall be increased to {0.75+0.011($\frac{N_b}{t}$)}	

Table 11.2b - I-beams and beams with restraint against web rotation

Type and position of loadings	Total web resistance, P_w
i) Single load or reaction  $c < 1.5d$. Load or reaction near or at free end	Stiffened and unstiffened flanges $P_w = t^2 c_7 p_y \left\{ 8.8 + 1.1\sqrt{\frac{N_b}{t}} \right\}$
ii) Single load or reaction  $c < 1.5d$. Load or reaction far from free end	Stiffened and unstiffened flanges $P_w = t^2 c_5 c_6 p_y \left\{ 13.2 + 2.87\sqrt{\frac{N_b}{t}} \right\}$
iii) Two opposite loads or reactions $e < 1.5d$  $c \leq 1.5d$. Loads or reactions near or at free end	Stiffened and unstiffened flanges $P_w = t^2 c_{10} c_{11} p_y \left\{ 8.8 + 1.1\sqrt{\frac{N_b}{t}} \right\}$
iv) Two opposite loads or reactions $e < 1.5d$  $c > 1.5d$. Loads or reactions far from free end	Stiffened and unstiffened flanges $P_w = t^2 c_8 c_9 p_y \left\{ 13.2 + 2.87\sqrt{\frac{N_b}{t}} \right\}$

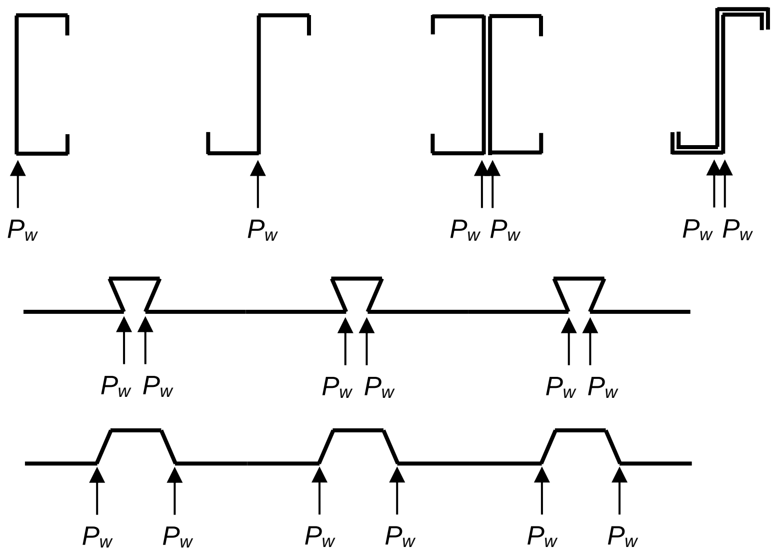


Figure 11.9 - Examples of cross-sections

In these relationships and the equations in Table 11.2a) and Table 11.2b):

d	is the overall web depth (mm);
t	is the web thickness (mm);
r	is the internal radius of corner (mm);
N_b	is the length of stiff bearing (mm); for the case of two equal and opposite concentrated loads distributed over unequal bearing lengths, the smaller value of N shall be taken;
P_w	is the web crushing capacity of a single web (N);
c	is the distance from the end of the section to the load or the reaction (mm);

$c_1 = 1.22 - 0.27\,k_w$ (11.36a)

$c_2 = 1.06 - 0.06\dfrac{r}{t} \leq 1.0$ (11.36b)

$c_3 = 1.33 - 0.40\,k_w$ (11.36c)

$c_4 = 1.15 - 0.15\dfrac{r}{t} \leq 1.0$ but ≥ 0.5 (11.36d)

$c_5 = 1.49 - 0.64\,k_w \geq 0.6$ (11.36e)

$c_6 = 0.88 + 0.12\,m_w$ (11.36f)

$c_7 = 1 + \dfrac{1}{750}\dfrac{d}{t}$ when $\dfrac{d}{t} < 150$; (11.36g)

$= 1.20$ when $\dfrac{d}{t} \geq 150$ (11.36h)

$c_8 = \dfrac{0.83}{k_w}$ when $\dfrac{d}{t} < 66.5$; (11.36i)

$= \dfrac{0.83\left(1.10 - \dfrac{1}{665}\dfrac{d}{t}\right)}{k_w}$ when $\dfrac{d}{t} \geq 66.5$ (11.36j)

$c_9 = 0.82 + 0.15\,m_w$ (11.36k)

$$c_{10} = \frac{0.83\left(0.98 - \frac{1}{865}\frac{d}{t}\right)}{k_w} \quad (11.36l)$$

$$c_{11} = 0.64 + 0.31\, m_w \quad (11.36m)$$

$$c_{12} = 0.7 + 0.3\,\left(\frac{\theta}{90}\right)^2 \quad (11.36n)$$

where

k_w $= \frac{p_y}{275}$ where p_y is the design strength (N/mm²) ; (11.37)

m_w $= \frac{t}{1.9}$; (11.38)

θ is the angle in degrees between the plane of web and the plane of bearing surface, where $45° \le \theta \le 90°$.

For built-up I-beams, or similar sections, the distance between the connector and the beam flange shall be kept as small as practicable.

11.4.5.2 Sheet profiles

The web crushing capacity of flat profile webs under concentrated forces, either loads or reactions, as shown in Figure 11.9, is given by

$$P_w = 0.15 c_o t^2 \sqrt{E\,p_y}\left(1 - 0.1\sqrt{\frac{r}{t}}\right)\left(0.5 + \sqrt{\frac{N_b}{50t}}\right)\left\{2.4 + \left(\frac{\theta}{90}\right)^2\right\} \quad (11.39)$$

where

c_o = 1.0 if the nearest edge of the concentrated force is located at a distance of not less than 1.5 d_w from the end of the sheet profile;
= 0.5 if the nearest edge of the concentrated force is located at a distance of less than 1.5 d_w from the end of the sheet profile;

d_w is the sloping distance between the intersection points of a web and the two flanges;
t is the net thickness of steel material;
r is the internal radius of corner;
N_b is the length of stiff bearing, which shall be at least 10 mm but not larger than 200 mm;
E is the modulus of elasticity;
p_y is the design strength of steel;
θ is the inclination of the web ($45° \le \theta \le 90°$).

11.4.6 Combined bending and web crushing

For flat webs of sections and sheet profiles subject to combined bending and web crushing action, the following equations should be satisfied:

a) $$\frac{F_w}{P_w} \le 1.0 \quad (11.40)$$

b) $$\frac{M}{M_c} \le 1.0 \quad (11.41)$$

c) $$1.2\left(\frac{F_w}{P_w}\right) + \left(\frac{M}{M_c}\right) \le 1.5$$ for sections having single thickness webs (11.42a)

or

$$1.1\left(\frac{F_w}{P_w}\right) + \left(\frac{M}{M_c}\right) \le 1.5$$ for sections having double thickness webs such as double C sections back-to-back, or similar sections with a high degree of rotational restraint to section webs (11.42b)

or

$$\left(\frac{F_w}{P_w}\right)+\left(\frac{M}{M_c}\right)\leq 1.25 \quad \text{for sheet profiles having single thickness webs} \qquad (11.42c)$$

where

F_w is the concentrated force;

P_w is the web crushing capacity;

M is the corresponding applied moment acting at the same cross section as F_w ;

M_c is the moment capacity.

11.4.7 Lateral buckling

11.4.7.1 General

Lateral buckling, also known as lateral-torsional or flexural-torsional buckling, in a member will occur if the member is not adequately restrained against lateral movement and longitudinal twisting. Non-linear finite element analysis allowing for imperfections and second-order effects could be used in place of the following effective length method.

Lateral restraints shall be considered to be fully effective if they are designed against a specific fraction, α, of the maximum force in the compression flange of a member where α is given as follows:

α = 3% when only one lateral restraint is attached to the member;
= 1.5% when two lateral restraints are attached to the member;
= 1% when three or more restraints are attached to the member.

Where several members share a common restraint, the lateral restraint should be designed against a total lateral force which is equal to the sum of the largest three.

A compound member composed of two sections in contact or separated back-to-back by a distance not greater than that required for an end gusset connection, shall be designed as a single integral member with an effective slenderness as defined in clause 11.4.7.2, provided that the main components are of a similar cross-section with their corresponding rectangular axes aligned and provided that they are interconnected properly at regular close intervals.

11.4.7.2 Effective lengths

For members susceptible to lateral torsional buckling, the effective length, L_E, shall be taken as follows:

a) For a member supported at both ends without any intermediate lateral restraint as shown in Figure 11.10, the effective length is given by:
   1) $L_E = 1.1\,L$ for a member free to rotate in all three directions, i.e. θ_1, θ_2 and θ_3 directions,
   2) $L_E = 0.9\,L$ for a member being restrained against torsional rotation θ_1 only,
   3) $L_E = 0.8\,L$ for a member being restrained against torsional rotation θ_1, and rotation about the minor axis θ_2,
   4) $L_E = 0.7\,L$ for a member being fully restrained against rotation in all three directions, i.e. θ_1, θ_2 and θ_3 directions.

   where L is the span of the member between end supports.

b) For a member attached to intermediate restraints through substantial connections to other steel members which is part of a fully framed structure, L_E shall be taken as 0.8 times the distance between intermediate restraints.

   For a member attached to intermediate restraints through less substantial connections, L_E shall be taken as 0.9 times the distance between intermediate restraints.

c) Where the length considered is the length between an end support and an intermediate restraint, the effective length coefficient shall be taken as the mean of the values obtained from a) and b) above.

d) In the case of a compound member composed of two C sections back-to-back designed as a single integral member with sufficient interconnections, the effective slenderness of the compound member $(L_E/r_y)_e$ shall be calculated as follows:

$$\left(\frac{L_E}{r_y}\right)_e = \sqrt{\left(\frac{L_E}{r_{cy2}}\right)^2 + \left(\frac{s}{r_{cy1}}\right)^2} \quad \geq 1.4\ \frac{s}{r_{cy}} \qquad (11.43)$$

where

L_E is the effective length of the compound member;

r_y is the radius of gyration of the compound member about the axis parallel to the webs allowing for the two sections acting as a single integral member;

r_{cy2} is the radius of gyration of the compound member about the axis parallel to the webs based on nominal geometric properties;

s is the longitudinal spacing between adjacent interconnections which shall not exceed 50 r_{cy};

r_{cy1} is the minimum radius of gyration of an individual C section.

The strength and the maximum spacing of interconnections shall comply with clause 11.6.4.

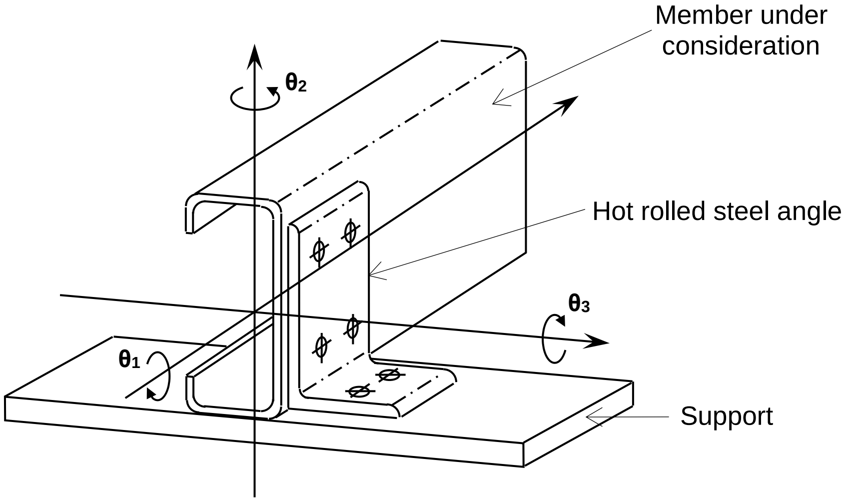


Figure 11.10 - Restraint condition for lateral buckling

11.4.7.3 Destabilizing loads

The condition of a destabilizing load exists when a load is applied to a member where both the load and the member are free to deflect laterally and twist longitudinally relative to the centroid of its cross section. In such cases, the effective lengths given in clause 11.4.7.2 should be increased by 20%.

11.4.7.4 Buckling moment resistance

11.4.7.4.1 Buckling resistance moment M_b

In each segment of a beam, the buckling resistance moment M_b shall satisfy the following equation:

$$m_{LT}\, M_x \leq M_b \qquad \text{and} \qquad M_x \leq M_c \qquad (11.44)$$

where

m_{LT} is the equivalent uniform moment factor for lateral torsional buckling of simple beams in Table 8.4, or conservatively taken as unity. For cantilever, m_{LT} is equal to 1.

M_x is the maximum bending moment along the beam segment.

M_b is the buckling moment resistance of a member with insufficient lateral restraint

$$= \frac{M_E M_Y}{\phi_B + \sqrt{\phi_B^2 - M_E M_Y}} \qquad \le M_c \qquad (11.45)$$

where

$$\phi_B = \frac{M_Y + (1+\eta)M_E}{2} \qquad (11.46)$$

M_c is the moment capacity of the member determined in accordance with clause 11.4.2.2;

M_Y is the elastic yield moment resistance of the member

$= p_y \times Z_{gc}$;

Z_{gc} is the elastic modulus of the gross cross-section with respect to the compression flange;

M_E is the elastic lateral buckling moment resistance determined in accordance with clause 11.4.7.4.2;

η is the Perry coefficient

$$= 0 \qquad \text{when } L_E / r_y \le 40 \qquad (11.47a)$$

$$= 0.002\left(\frac{L_E}{r_y} - 40\right) \qquad \text{when } L_E / r_y > 40 \qquad (11.47b)$$

where

L_E is the effective length determined in accordance with clause 11.4.7.2;

r_y is the radius of gyration of the member about the y axis.

11.4.7.4.2 Determination of M_E

The elastic lateral buckling moment resistance, M_E, for members loaded effectively through the shear centre of their cross sections shall be determined as follows:

a) for members of single C sections, compound C sections back-to-back and equal flange I-section bent in the plane of the web:

$$M_E = \frac{\pi^2 AEd}{2(L_E / r_y)^2} C_{tw} \qquad (11.48a)$$

If the member is torsionally restrained at the cross-sections at both the load application and the support points, it should be considered to be loaded through the shear centre for determination of M_E.

b) for members of Z sections bent in the plane of the web:

$$M_E = \frac{\pi^2 AEd}{4(L_E / r_y)^2} C_{tw} \qquad (11.48b)$$

c) for members of T sections bent in the plane of the web:

$$M_E = \frac{\pi^2 AEd}{2(L_E / r_y)^2} C_T \left[\overline{C}_{tw} + 1\right] \qquad \text{when the flanges are in compression} \qquad (11.48c)$$

$$= \frac{\pi^2 AEd}{2(L_E / r_y)^2} C_T \left[\overline{C}_{tw} - 1\right] \qquad \text{when the flanges are in tension} \qquad (11.48d)$$

where

A is the cross-sectional area of the member;

E is the modulus of elasticity;

d is the overall web depth;

$$C_{tw} = \sqrt{\left\{1 + \frac{1}{20}\left(\frac{L_E}{r_y}\frac{t}{d}\right)^2\right\}} \quad ; \qquad (11.49a)$$

$$\bar{C}_{tw} = \sqrt{\left\{1+\frac{1}{20}\left(\frac{1}{C_T}\frac{L_E}{r_y}\frac{t}{d}\right)^2\right\}} \;; \qquad (11.49b)$$

$$C_T = \frac{1+1.5\frac{B}{d}-0.25(\frac{B}{d})^3}{1+2\frac{B}{d}} \;; \qquad (11.49c)$$

where

B is the total width of the flange of a T-section;

t is the net thickness of steel material;

L_E and r_y are as defined in clause 11.4.7.4.1.

It should be noted that if a negative value of C_T is obtained, the member should be regarded as fully restrained.

C_{tw} may conservatively be taken as 1.0 for members in a) and b) above.

11.4.8 Calculation of deflection

11.4.8.1 General

Deflections should be calculated using elastic analysis. Due allowance shall be made for the effects of non-uniform loading. The effective cross section for deflection calculations shall be determined in accordance with clause 11.3.4.7. In the absence of more accurate analysis and information, the effective second moment of area I_{ser} of open section and sheet profiles shall be assumed to be constant throughout each span.

Recommended deflection limits are given in clause 5.2.

11.4.8.2 Single spans

For a uniformly loaded single span, the deflection δ is given by

$$\delta = \frac{5}{384}\frac{w\,L^4}{E\,I_{ser}} \qquad (11.50)$$

where

w is the intensity of loading at serviceability limit state;

L is the span between centres of supports;

I_{ser} is the effective second moment of area of the open section and sheet profiles at serviceability limit state, determined at midspan.

11.4.8.3 Multi-spans

11.4.8.3.1 Type of loading

When calculating deflections due to imposed gravity loads, the possibility of pattern loading between different spans should be considered.

However when calculating deflections of sheet profiles used as permanent shuttering for slabs, the weight of the wet concrete should be taken as uniformly distributed on all spans.

Uniform loading on all spans should also be taken when calculating deflections of cladding and roof decking subject to wind load only.

11.4.8.3.2 Calculation of deflection

Unless a more detailed analysis is undertaken, the following approximations may be assumed to cover the extent of loading most likely to be met in practice, providing that the spans do not vary by more than 15% of the greatest spans.

The maximum deflection due to uniformly distributed load on all spans is given by

$$\delta = \frac{1}{185}\frac{w\,L^4}{E\,I_{ser}} \qquad (11.51)$$

The maximum deflection δ due to pattern loading is given by

$$\delta = \frac{3}{384}\frac{w L^4}{E I_{ser}} \qquad (11.52)$$

where

L is the greatest span between centres of supports.

11.4.9 Effects of torsion

For members of open sections, the effects of torsion should be avoided whenever possible either by providing sufficient restraints designed to resist twisting or by ensuring that all lateral loads are applied through the shear centres of the members.

11.4.9.1 Direct stresses due to combined bending and torsion

For members subjected to combined bending and torsion, the maximum stress due to both effects combined, determined on the basis of the gross section and the unfactored loads, shall not exceed the design strength, p_y.

11.4.9.2 Angle of twist

The angle of twist of a member which is subject to torsion shall not be so great as to change significantly the shape of the cross-section or its capability to resist bending.

11.5 MEMBERS UNDER AXIAL LOADS

11.5.1 Members under tension

In general, advanced analysis, second-order analysis or the non-linear finite element analysis allowing for imperfections and second-order effects could be used in place of the following effective length method.

11.5.1.1 Tensile capacity

The tensile capacity, P_t, of a member is given by:

$$P_t = A_{net}\, p_y \qquad (11.53)$$

where

p_y is the design strength.

A_{net} is the effective net area of a section or a sheet profile

$$= A_c + A_u \frac{A_c}{(A_c + A_u/3)} \qquad (11.54a)$$

for single angles connected through one leg only, single C and Z sections connected only through the web, and T sections connected only through the flange, or

$$= A_c + A_u \frac{A_c}{(A_c + A_u/5)} \qquad (11.54b)$$

for double angles, C and T sections connected back-to-back

where

A_c is the net sectional area of the connected leg;

A_u is the gross sectional area of the unconnected leg or legs.

It should be noted that

a) for double angles connected to one side of a gusset or section, the angles shall be designed individually.

b) for double angles, C and T sections connected back-to-back, the two components shall be

   1) in direct contact, or separated by solid packing pieces by a distance not exceeding the sum of the thickness of the parts; and

2) inter-connected by bolts such that the slenderness of the individual components does not exceed 80.

These rules only apply when the width to thickness ratios of the unconnected elements are less than 20. For width to thickness ratios greater than 20, the nominal moment due to eccentricity of applied forces should be taken into account.

In determining the net gross area of a connected leg, the area to be deducted from the gross sectional area should be the maximum sum of the sectional areas of the holes in any cross-section at right angles to the direction of stress in the member.

However, when the holes are staggered, the area to be deducted should be the greater of:

a) the deduction for non-staggered holes;

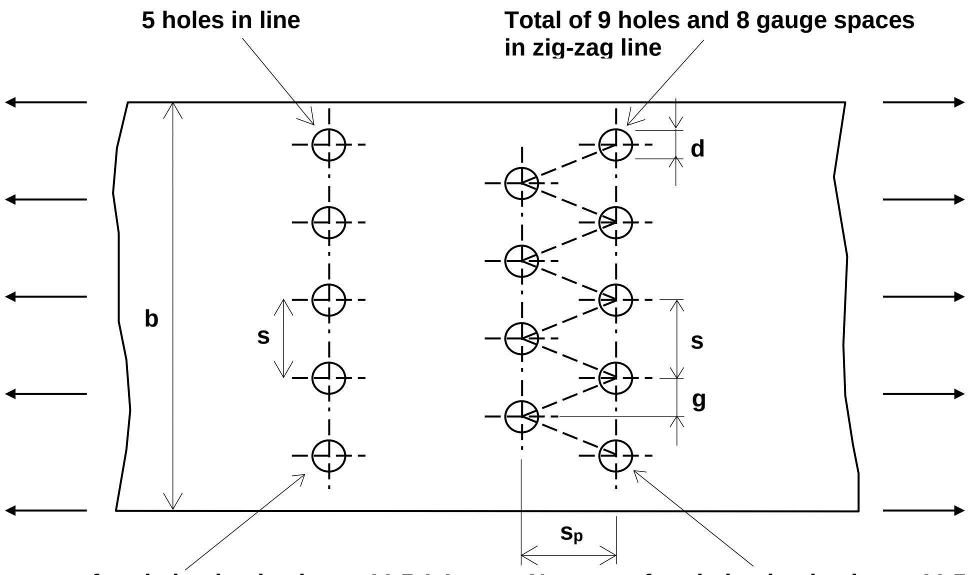
b) the sum of the sectional areas of all holes in any zigzag line extending progressively across the member or part of the member, less $s_p^2\ t\ /\ 4\ g$ for each gauge space in the chain of holes.

where

s_p is the staggered pitch, i.e. the distance measured parallel to the direction of stress in the member centre-to-centre of holes in consecutive lines (see Figure 11.11);

t is the net thickness of the steel material;

g is the gauge, i.e. the distance measured at right angles to the direction of stress in the member centre-to-centre of holes in consecutive lines (see Figure 11.11).



Net area after deduction in clause 11.5.1.1
= $b\ t - 5\ d\ t$

Net area after deduction in clause 11.5.1.1

$$= b\ t - \left(9\ d\ t - \frac{8\ s_p^{\ 2}\ t}{4\ g} \right)$$

Figure 11.11 - Nomenclature for staggered holes with example

11.5.1.2 *Combined tension and bending*

For members subject to combined tension and bending, the following equation shall be satisfied:

a) $\frac{F_t}{P_t} \leq 1$ (11.55)

b) $\frac{M_x}{M_{cx}} \leq 1$ (11.56)

c) $\frac{M_y}{M_{cy}} \leq 1$ (11.57)

d) $\frac{F_t}{P_t} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$ (11.58)

where

F_t is the applied tensile force

P_t is the tensile capacity determined in accordance with clause 11.5.1.1;

M_x is the applied moment about the x axis;

M_y is the applied moment about the y axis;

M_{cx} is the moment capacity under pure bending about the x axis; and

M_{cy} is the moment capacity under pure bending about the y axis.

11.5.2 Members under compression

11.5.2.1 *Compressive capacity*

The compressive capacity, P_{cs}, of a member is given by:

$P_{cs} = A_e \ p_y$ (11.59)

where

p_y is the design strength;

A_e is the area of the effective cross section of the member under compression determined after full consideration against local buckling to clause 11.3.4.

11.5.3 Flexural buckling

11.5.3.1 *Effective lengths*

The effective length of a member in compression shall be established in accordance with clause 8.7.2 and Table 8.6 or on the basis of good engineering practice.

11.5.3.2 *Maximum slenderness*

In general, the slenderness ratio shall be taken as the effective length, L_E, divided by the radius of gyration of the gross cross-section about the relevant axis, r . The maximum value of the slenderness ratio, L_E / r, shall not exceed:

a) 180 for members resisting loads other than wind loads;

b) 250 for members resisting self weight and wind loads only;

c) 350 for any member acting normally as a tie but subject to reversal of stress resulting from the action of wind.

11.5.3.3 *Ultimate loads*

For members with cross sections symmetrical about both principal axes or closed cross-sections which are not subject to flexural-torsional buckling, or members which are braced against twisting, the compressive buckling resistance, P_c, is given by:

$$P_c = \frac{P_E P_{cs}}{\phi + \sqrt{\phi^2 - P_E P_{cs}}} \qquad (11.60)$$

where

$$\phi = \frac{P_{cs} + (1+\eta)P_E}{2} \tag{11.61}$$

where

P_E is the minimum elastic buckling load

$$= \frac{\pi^2 EI}{{L_E}^2} \tag{11.62}$$

where

E is the modulus of elasticity;

I is the second moment of area of the cross-section about the critical axis;

L_E is the effective length of the member about the critical axis;

η is the Perry coefficient, such that

$= 0$ for $L_E / r \leq 20$ (11.63a)

$= 0.002\ (L_E / r - 20)$ for $L_E / r > 20$ (11.63b)

r is the radius of gyration of the gross cross-section corresponding to P_E.

It should be noted that for members susceptible to flexural-torsional buckling, P_E shall be multiplied with the flexural-torsional buckling parameter α_{TF} which is determined in accordance with clause 11.5.4.1. For bi-symmetrical cross sections, α_{TF} is equal to unity.

11.5.3.4 *Singly symmetrical sections*

For members with cross sections symmetrical about only a single axis and which are not subject to flexural-torsional buckling, or which are braced against twisting, the effects of the shift of the neutral axis shall be taken into account in evaluation of the maximum compressive resistance.

The shift of the neutral axis shall be calculated by determining the neutral axis position of the gross cross-section and that of the effective cross-section. In evaluation of the neutral axis position of the effective cross-section, the effective portions shall be positioned as detailed in clause 11.3.3 and shown in Figure 11.12.

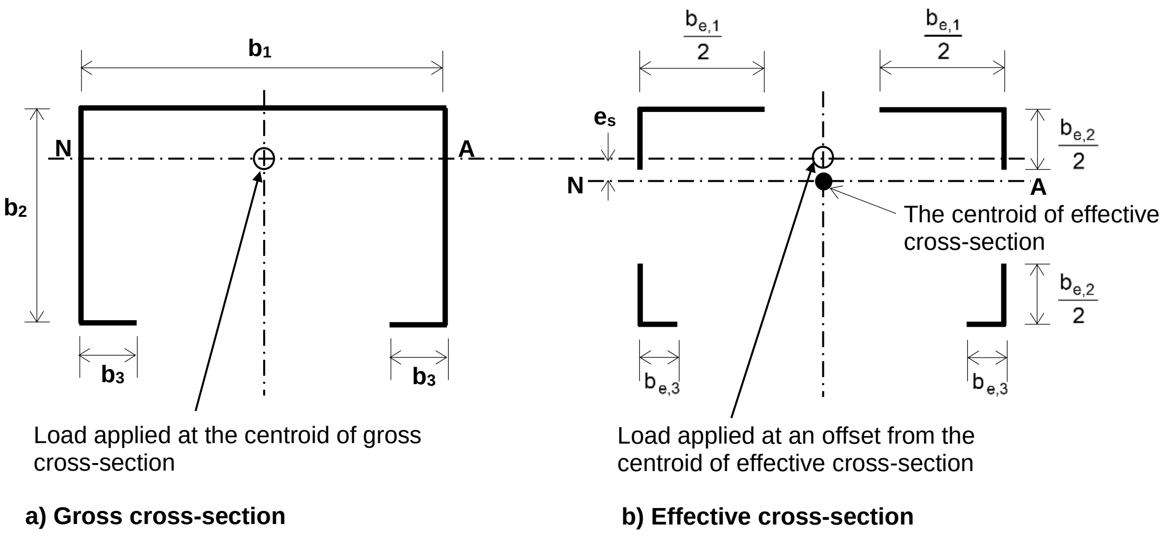


Figure 11.12 - Compression of singly symmetrical section

The modified compressive buckling resistance, P'_c, of the member is then given by

$$P'_c = \frac{M_c}{(M_c + P_c e_s)} \; P_c \qquad (11.64)$$

where

M_c is the moment capacity determined in accordance with clause 11.4.2.2, having due regard to the direction of moment application as indicated in Figure 11.12;

P_c is the compressive buckling resistance of the member determined in accordance with clause 11.5.3.3;

e_s is the distance between the neutral axis of the gross cross-section and that of the effective cross-section, as indicated in Figure 11.12.

11.5.3.5 *Compound members composed of C sections back-to-back*

A compound member composed of two C sections in contact or separated back-to-back by a distance not greater than that required for an end gusset connection, should be designed as a single integral member subject to the following conditions:

a) The C sections are similar in cross-section with their corresponding rectangular axes aligned.

b) The C sections are interconnected with structural fasteners in accordance with clause 11.6.

c) The effective slenderness of the compound member $(L_E / r_y)_e$ about the axis parallel to the webs of the C sections, is given by

$$\left(\frac{L_E}{r_y}\right)_e = \sqrt{\left(\frac{L_E}{r_{cy2}}\right)^2 + \left(\frac{s}{r_{cy1}}\right)^2} \geq 1.4 \frac{s}{r_{cy1}} \qquad (11.65)$$

where

L_E is the effective length of the compound member;

r_y is the radius of gyration of the compound member about the axis parallel to the webs allowing for the two sections acting as a single integral member;

r_{cy2} is the radius of gyration of the compound member about the axis parallel to the webs based on nominal geometric properties;

s is the longitudinal spacing between adjacent interconnections which shall not exceed 50 r_{cy1} ;

r_{cy1} is the minimum radius of gyration of an individual C section.

The strength and the maximum spacing of interconnections is given in clause 11.6.

11.5.4 Flexural-torsional buckling

The design procedure given in clause 11.5.4.1 applies only to members which are braced in both the x and the y directions at the ends or at the same points of supports of the members.

11.5.4.1 Sections with at least one axis of symmetry (x axis)

For members which have at least one axis of symmetry, taken as the x axis, and which are subject to flexural-torsional buckling, and designed according to clause 11.5.3, the value of α_{TF} is given by:

for $P_E \le P_{TF}$ $\qquad \alpha_{TF} = 1$ (11.66a)

for $P_E > P_{TF}$ $\qquad \alpha_{TF} = \sqrt{\dfrac{P_E}{P_{TF}}}$ (11.66b)

where

P_E is the elastic flexural buckling load for a member under compression:

$$= \frac{\pi^2 EI}{L_E^{\,2}} \qquad (11.67)$$

where

E is the modulus of elasticity;

I is the second moment of area about the y axis;

L_E is the effective length corresponding to the minimum radius of gyration;

P_{TF} is the flexural-torsional buckling load of a member under compression:

$$= \frac{1}{2\beta}\left[(P_{Ex} + P_T) - \sqrt{(P_{Ex} + P_T)^2 - 4\beta P_{Ex} P_T}\right] \qquad (11.68)$$

where

P_{Ex} is the elastic flexural buckling load for the member about the x axis:

$$= \frac{\pi^2 EI_x}{L_E^{\,2}} \qquad (11.69)$$

P_T is the elastic torsional buckling load of the member:

$$= \frac{1}{r_o^{\,2}}\left(GJ + \frac{2\,\pi^2 EC_w}{L_{EZ}^{\,2}}\right) \qquad (11.70)$$

$$\beta \quad = 1 - \left(\frac{e_{sc}}{r_o}\right)^2 \qquad (11.71)$$

L_{EZ} is the effective length of the member against torsion and warping;

r_o is the polar radius of gyration about the shear centre:

$$= \sqrt{r_x^{\,2} + r_y^{\,2} + e_{sc}^{\,2}} \qquad (11.72)$$

r_x, r_y are the radii of gyration about the x and y axes;

G is the shear modulus;

e_{sc} is the distance from the shear centre to the centroid measured along the x axis;

J is the St Venant torsion constant

$$= \sum \frac{bt^3}{3}$$

where b is the element flat width and t is the net thickness of the steel material;

I_x is the second moment of area about the x axis;

C_w is the warping constant of the cross-section.

11.5.4.2 Non-symmetrical sections

For members with non-symmetrical cross-sections, the maximum load shall be determined either by advanced analysis or by testing.

11.5.5 Combined compression and bending

In general, advanced analysis, second-order analysis or the non-linear finite element analysis allowing for imperfections and second-order effects could be used in place of the following effective length method.

When designed by the effective length method, members under combined compression and bending shall be checked for local capacities and overall buckling resistances.

Clauses 11.5.5.1 and 11.5.5.2 apply to members which have at least one axis of symmetry and which are not subject to torsional or flexural-torsional buckling.

11.5.5.1 Local capacity check

The following check against local capacities shall be satisfied at various points along the length of the members:

$$\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \le 1 \qquad (11.73)$$

where

F_c is the applied compression force;

P_{cs} is the compression capacity;

M_x is the applied moment about the x axis;

M_y is the applied moment about the y axis;

M_{cx} is the moment capacity under pure bending about the x axis in accordance with clause 11.4.2.2;

M_{cy} is the moment capacity under pure bending about the y axis in accordance with clause 11.4.2.2.

11.5.5.2 Overall buckling check

a) For members not subject to lateral buckling, the following simplified check shall be satisfied:

$$\frac{F_c}{P_c} + \frac{m_x\, M_x}{M_{cx}} + \frac{m_y\, M_y}{M_{cy}} \le 1 \qquad (11.74)$$

b) For members subject to lateral buckling, the following simplified check shall be satisfied:

$$\frac{F_c}{P_{cy}} + \frac{m_{LT}\, M_{LT}}{M_b} + \frac{m_y\, M_y}{M_{cy}} \le 1 \qquad (11.75)$$

where

P_c is the compressive buckling resistance;

M_b is the buckling moment resistance about the x (major) axis as defined in clause 11.4.7.4;

F_c, M_x, M_{cx}, M_y and M_{cy} are as defined in clause 11.5.5.1.

The magnitudes of moments M_x and M_y shall take into account any moment induced by the shift of the neutral axis caused by the compression force.

m_{LT} is the equivalent uniform moment factor for lateral torsional buckling over the segment length L_{LT} governing M_b from Table 8.4;

m_x is the equivalent uniform moment factor for major axis flexural buckling over the segment length L_x governing P_{cx} from Table 8.9;

m_y is the equivalent uniform moment factor for minor axis flexural buckling over the segment length L_y governing P_{cy} from Table 8.9;

L_{LT} is the segment length between restraints against lateral torsional buckling;

L_x is the segment length between restraints against flexural buckling about the major axis;

L_y is the segment length between restraints against flexural buckling about the minor axis.

For members subject to significant P-Δ-δ effect, refer to clause 8.9.2.

11.6 CONNECTIONS

11.6.1 General recommendations

This clause provides detailed design recommendations on connections and fastenings with the following types of fasteners:

- Bolts
- Screws and blind rivets

Design of interconnections in forming an I-section using two C sections back-to-back and bolted moment connections of an I-section using two C sections back-to-back are also provided.

In general, connections and fastenings shall be designed using a realistic assumption of the distribution of internal forces, taking into account of relative stiffnesses. This distribution shall correspond with direct load paths through the elements of connections. It is essential that equilibrium with the applied forces is maintained. Ease of fabrication and erection shall be considered in the design of joints and splices. Attention shall be paid to clearances necessary for tightening of fasteners, subsequent inspection, surface treatment and maintenance.

As ductility of steel assists the distribution of local forces generated within a joint, residual stresses and stresses due to tightening of fasteners with normal accuracy of fit-up shall be ignored.

11.6.1.1 Intersections

Usually, members meeting at a joint shall be arranged with their centroidal axes meeting at a point. Where there is eccentricity at intersections, both the members and the connections shall be designed to accommodate the resulting moments. In the case of bolted framing of angles and tees, the setting-out lines of the bolts shall be adopted instead of the centroidal axis.

11.6.1.2 Strength of individual fasteners

The strength of individual fasteners shall be calculated in accordance with clause 11.6.2 or 11.6.3, or determined by testing.

11.6.1.3 Forces in individual fasteners

The shear forces on individual fasteners in a connection shall be assumed to be equal provided that the material is less than or equal to 4 mm thick. Otherwise the shear forces on individual fasteners shall be calculated by elastic analysis.

11.6.2 Fastenings with bolts

The recommendations in this clause apply to bolts with a diameter d in the range:

$$10 \text{ mm} \leq d \leq 30 \text{ mm} \tag{11.76}$$

The recommendations are applicable to bolts in nominally 2 mm oversize clearance holes.

11.6.2.1 Effective diameter and areas of bolts

The tensile stress area of the bolt, A_t , shall be used in determining the shear and the tensile capacities of a bolt. For bolts without a defined tensile stress area, A_t shall be taken as the area at the bottom of the threads.

Where it is shown that the threads do not occur in the shear plane, the shank area, A, shall be used in the calculation of shear capacity.

In the calculation of thread length, allowance shall be made for tolerance and thread run off.

11.6.2.2 Shear and tension capacities of bolts

The shear capacity, P_s, of a bolt is given by:

$$P_s = p_s A_s \tag{11.77}$$

The tension capacity, P_t, of a bolt is given by:

$$P_t = p_t A_t \tag{11.78}$$

where

A_s is A_t or A as appropriate as defined in clause 11.6.2.1;
p_s is the shear strength given in Table 11.3;
p_t is the tensile strength given in Table 11.3.

Table 11.3 - Design strength of bolts in clearance holes

	General grade of bolts	Common bolt grade	
		M4.6	**M8.8**
Shear strength, p_s (N/mm^2)	0.48 U_{fb} but ≤ 0.69 Y_{fb}	160	375
Tensile strength, p_t (N/mm^2)	0.58 U_{fb} but ≤ 0.83 Y_{fb}	195	450

Notes: Y_{fb} is the specified minimum yield strength of bolts;
U_{fb} is the specified minimum tensile strength of bolts.

11.6.2.3 *Combined shear and tension*

When bolts are subject to combined shear and tension, the following relationship shall be satisfied in addition to the recommendations in clause 11.6.2.2:

$$\frac{V}{V_c}+\frac{F_t}{P_t} \le 1.4 \qquad (11.79)$$

where

V is the applied shear force;
F_t is the applied tension force;
V_c is the shear capacity of bolt determined in accordance with clause 11.6.2.2;
P_t is the tension capacity of bolt determined in accordance with clause 11.6.2.2.

11.6.2.4 *Minimum pitch, and minimum edge and end distances*

For steel materials less than or equal to 4 mm thick, the distance between the centres of adjacent bolts in the line of force, or the bolt pitch, shall not be less than 3*d*, where *d* is the diameter of the bolt. For steel materials greater than 4 mm thick, the bolt pitch shall not be less than 2.5*d*.

The distance between the centre of a bolt and any edge of the connected member, i.e. the edge distances and the end distances shall not be less than 1.5*d*.

11.6.2.5 *Bearing capacity*

The bearing capacity, P_{bs} , of connected elements for each bolt in the line of force is given by:

$$P_{bs} = \alpha \, \rho \, d \, t \, p_y \qquad (11.80)$$

where

α is the strength coefficient which is given as follows:

i) for $t \le 1$ mm

$$\alpha = 2.1 \qquad (11.81a)$$

ii) for $1 < t \le 3$ mm

$$\alpha = 2.1+\left(0.3\frac{d_e}{d}-0.45\right)(t-1) \quad \text{when} \quad \frac{d_e}{d} < 3 \qquad (11.81b)$$

$$= 1.65+0.45t \quad \text{when} \quad \frac{d_e}{d} \ge 3 \qquad (11.81c)$$

iii) for $3 < t \le 8$ mm

$$\alpha = 1.2+0.6\frac{d_e}{d} \quad \text{when} \quad \frac{d_e}{d} < 3 \qquad (11.81d)$$

$$= 3.0 \quad \text{when} \quad \frac{d_e}{d} \ge 3 \qquad (11.81e)$$

ρ = 1.0 when washers are used under both the bolt heads and the nuts;
= 0.75 when only a single washer or no washer is used;
d is the nominal diameter (mm);

t is the net thickness of the steel material (mm);
p_y is the design strength (N/mm^2); and
d_e is the distance from the centre of a bolt to the end of the connected element in the direction of the bolt force (mm).

11.6.2.6 Tensile strength on net section

The tensile strength, p_t, of the net area of an element in a bolted connection is given by:

$$p_t = p_y \quad \text{or} \tag{11.82a}$$

$$= (0.1 + 3\frac{d}{s})\, p_y \tag{11.82b}$$

where:

p_y is the design strength (N/mm^2);
d is the diameter of the bolt (mm);
s is the distance between centres of bolts normal to the line of force (see Figure 11.11) or, where there is only a single line of bolts, the width of sheet (mm).

11.6.2.7 Prying

In connections subject to tension, prying action should be ignored provided that the design strengths of bolts given in Table 11.3 are used.

11.6.3 Fastenings with screws and blind rivets

This clause applies to self-tapping screws, including thread-forming, thread-cutting or self-drilling screws, and to blind rivets with a diameter d in the range of:

$$3.0 \text{ mm} \leq d \leq 7.5 \text{ mm} \tag{11.83}$$

If components of different thickness are connected, the head of the screw or the preformed head of the rivet shall be in contact with the thinner component.

The diameter of pre-drilled holes shall follow strictly in accordance with the manufacturer's recommendations.

Both the shear capacity, P_{fs}, and the tensile capacity, P_{ft}, of screws and rivets shall be determined by testing or provided by manufacturer. In order to avoid brittle failure, the size of the fastener shall be such that P_{fs} is not less than 1.25 P_s and P_{ft} is not less than 1.25 P_t where P_s and P_t are the shear and the tensile forces acting onto the fastener.

11.6.3.1 Minimum pitch, and minimum edge and end distances

The distance between centres of fasteners shall be not less than 3d.

The distance from the centre of a fastener to the edge of any part shall not be less than 3d.

If the connection is subjected to force in one direction only, such as to cause shear force in the fastener, the minimum edge distance shall be reduced to 1.5d or 10 mm, whichever is the smaller, in the direction normal to the line of force.

11.6.3.2 Capacity against shear force

The shear capacity, P_s, of a screw or a rivet in tilting and bearing is given by:

a) for $\frac{t_4}{t_3} = 1.0$,

$$P_s = 3.2\sqrt{t_3^{\,3}\, d}\; p_y \quad \leq \quad 2.1\, t_3\, d\, p_y \tag{11.84a}$$

b) for $\frac{t_4}{t_3} \geq 2.5$,

$$P_s = 2.1\, t_3\, \mathrm{d}\, p_y \tag{11.84b}$$

For $1.0 < \frac{t_4}{t_3} < 2.5$, P_s shall be determined by linear interpolation between the results obtained from a) and b) above.

where

t_3 is the thickness of the element in contact with the screw head or the preformed rivet head;

t_4 is the thickness of the element remote from the screw head or the preformed rivet head;

d is the diameter of the fastener;

p_y is the design strength of the element material.

11.6.3.3 Capacity against tensile force

For screws which carry significant tensile forces, the head of the screw, or the washer, if present, shall have an overall diameter d_w of at least 8 mm and shall have adequate rigidity. However, blind rivets shall not be used to carry significant tensile forces.

The tensile capacity P_t of a screwed connection shall be taken as the smallest of the following:

a) pulling of the connected element over the screw head or washer:

For connected element of thickness t_1 less than 2.0 mm and washer size d_w less than 25 mm,

$$P_t = 1.1\, t_1\, d_w\, p_y \qquad (11.85)$$

where

d_w is the diameter of the washer;

t_1 is the thickness of the element in contact with the screw head or the preformed rivet head. For other configurations, the tensile capacity shall be determined by testing.

b) pull out from the base element:

For connected element of thickness t_2 larger than 0.9 mm,

$$P_t = 0.65\, d\, t_2\, p_y \qquad (11.86)$$

where

t_2 is the thickness of the element remote from the screw head or preformed rivet head.

11.6.4 Interconnections in compound members

11.6.4.1 Maximum pitch: compression members

The distance between centres, in the line of force, of fasteners connecting a compression cover plate or sheet to a non-integral stiffener or other element shall not exceed any of the following:

a) the spacing required to transmit the shear force between the connected parts;

b) 37 $t\,\varepsilon$ where t is the thickness of the cover plate or sheet in mm, and $\varepsilon = \sqrt{\frac{275}{Y_s}}$

where Y_s is the yield strength of the cover plate or sheet in N/mm^2;

c) three times the flat width of the narrowest unstiffened compression element in that portion of the cover plate or sheet which is adjacent to the connection, or 30 $t\,\varepsilon$ whichever is greater.

11.6.4.2 Maximum pitch: connections of two C sections back-to-back to form an I-section

For compound members composed of two C sections back-to-back, interconnected by structural fasteners, either the individual sections shall be designed between points of interconnection in accordance with clauses 11.4 and 11.5 as appropriate, or the compound member shall be designed as a single integral member on the basis of an effective slenderness as defined in clause 11.4.7.2d) provided the longitudinal spacing s of the interconnections complies with the following.

a) For a compression member, designed in accordance with clause 11.5, at least two fasteners shall be provided in line across the width of all members that are sufficiently wide to accommodate them. Moreover, the spacing of interconnections,

s, shall be such that

i) the member length is divided into at least three parts of approximately equal length;

ii) $s \leq 50\, r_{cy1}$ (11.87)

where

s is the longitudinal spacing of interconnection;

r_{cy1} is the minimum radius of gyration of an individual C section.

The interconnecting structural fasteners shall be designed to transmit the longitudinal shear force, F_s, between the C sections induced by a transverse shear force, V, at any point in the compound member. The value of V shall be taken as not less than 2.5% of the design axial force in the compound member plus any load due to its self weight or wind load. The resulting longitudinal shear force per interconnection is given by:

$$F_s = \frac{V}{4}\left(\frac{s}{r_{cy1}}\right) \tag{11.88}$$

where

s / r_{cy1} is the local slenderness of an individual C section as given in clause 11.4.7.2d).

b) For a flexural member designed in accordance with clause 11.4, at least two structural fasteners shall be provided in line across the width of all members as shown in Figure 11.13. The tendency of the individual C sections to separate by twisting shall be resisted by limiting the spacing of interconnection, s , such that:

i) the member length is divided into at least three parts of approximately equal length;

ii) $s \leq 50\, r_{cy1}$ (11.89)

where

s is the longitudinal spacing of interconnections;

r_{cy1} is the minimum radius of gyration of an individual C section.

iii) the tensile capacity, P_t of the individual interconnection is greater than the induced transverse shear force, F_s :

$P_t \geq F_s$

where

$$F_s = \frac{F\,e}{2\,h} \tag{11.90}$$

e is the distance between the shear centre of the C section and the mid-plane of the web;

h is the vertical distance between the two rows of fasteners near or at the top and the bottom flanges;

F is the local concentrated force or the reaction force between the points of interconnection under consideration; or, for distributed load $F = w\,s$;

w is the load intensity on the member acting on a bearing length of s/2 each side of the interconnection under consideration.

The maximum spacing of interconnections depends on the load intensity acting at the connection. Therefore, if uniform spacing of connections is used over the whole length of the member, it shall be determined at the point of maximum local load intensity. If, however, this procedure results in uneconomically close spacing, then either the spacing shall be varied along the member length according to the variation of the load intensity, or reinforcing cover plates shall be provided to the flanges at the points where concentrated loads or reaction forces occur. The shear strength of the connections joining such plates to the flanges shall then be used for P_t, and h in the equation should be taken as the depth of the beam.

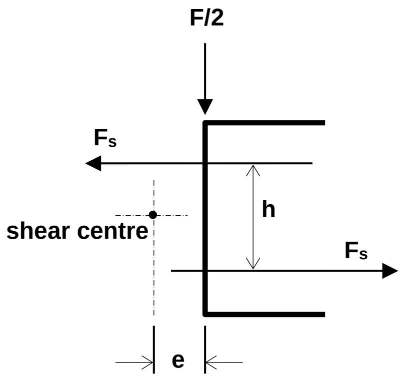


Figure 11.13 - Connection forces in back-to-back members

11.6.5 Bolted moment connections of compound members

For compound members composed of two C sections back-to-back with sufficient interconnections, bolted moment connections using fabricated inverted T sections as column bases and gusset plates as beam-column connections are shown in Figure 11.14.

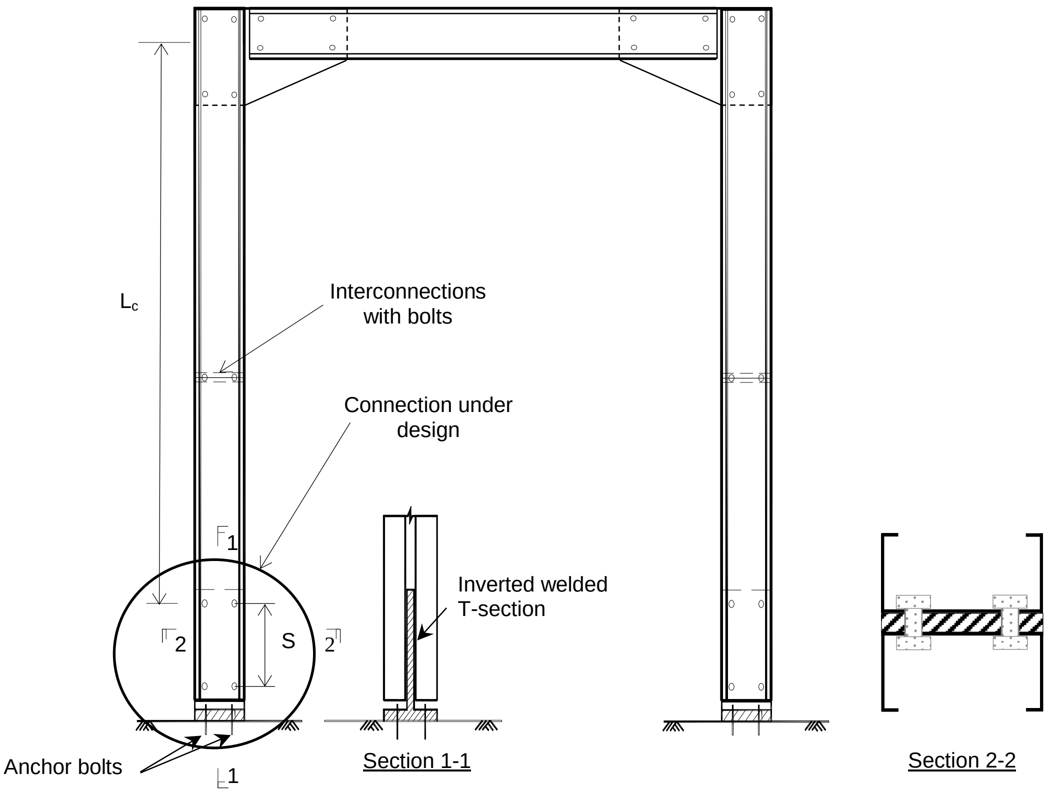


Figure 11.14 - Typical bolted moment connection

Both fabricated T-section and anchor bolt shall be designed in accordance with clause 9.4 while the force distribution within the bolted connections shall be determined through force and moment equilibrium consideration. The design expressions for both the forces and the moments in the bolted moment connections of both the column bases and the beam-column joints shown in Figure 11.14 are given in Figure 11.15.

For connected sections subject to both bending and shear, the following equations shall be satisfied:

a) Shear resistance

$$v_U = \frac{V_U}{V_{c,U}} \leq 1.0 \quad ; \qquad v_L = \frac{V_L}{V_{c,L}} \leq 1.0 \qquad \text{(11.91a \& 11.91b)}$$

b) Moment resistance

$$m_U = \frac{M_U}{M_{c,U}} \leq 1.0 \quad ; \qquad m_L = \frac{M_L}{M_{c,L}} \leq 1.0 \qquad \text{(11.92a \& 11.92b)}$$

c) Combined bending and shear

$$1.25\, v_U^2 + 1.25\, m_U^2 \leq 1.0 \qquad \text{(11.93a)}$$

$$1.25\, v_L^2 + 1.25\, m_L^2 \leq 1.0 \qquad \text{(11.93b)}$$

where

v_U , v_L are the shear force ratios at the upper and the lower levels of the critical cross-section respectively ;

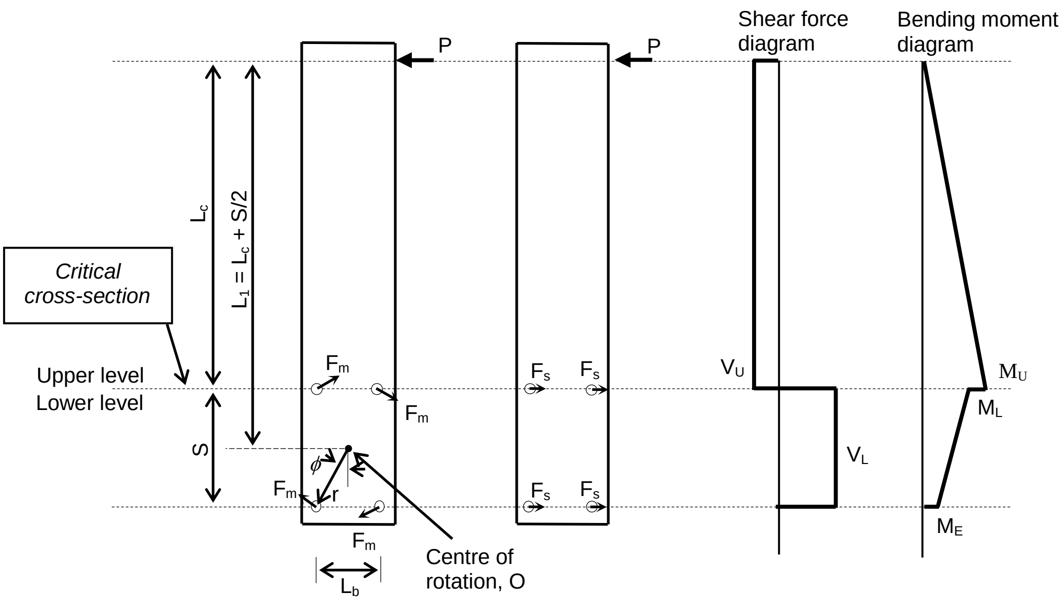
m_U , m_L are the moment ratios at the upper and the lower levels of the critical cross-section respectively ;

$V_{c,U}$, $V_{c,L}$ are the design shear capacities at the upper and the lower levels of the critical cross-section respectively; and

$M_{c,U}$, $M_{c,L}$ are the design moment capacities at the upper and the lower levels of the critical cross-section respectively.

It should be noted that as the C sections below the critical cross-section are firmly attached to the webs of the gusset plate through bolts, local buckling is unlikely to occur, and hence, $V_{c,L}$ shall be taken as the plastic shear capacity of the C section while $M_{c,L}$ shall be taken as the moment capacity of the gross cross sections. Moreover, the restraining effect due to the presence of the gusset plate on shear buckling of the connected sections above the critical cross-section shall be allowed for.

The presence of bolt holes in the cross-sections shall be allowed for during the determination of both the shear and the moment resistances.



Bolt force due to moment, F_m

$$P \times L_1 = 4 \times F_m \times r$$

$$\Rightarrow F_m = \frac{R\,L_1}{4\,r}$$

$$F_{m,h} = F_m \times \cos\phi$$

$$F_{m,v} = F_m \times \sin\phi$$

Bolt force due to shear, F_s

$$F_s = P / 4$$

$$V_U = P$$

$$V_L = 2 \times (F_s - F_{m,h})$$

$$M_U = P \times L_c$$

$$M_L = M_U - F_{m,v} \times L_b$$

$$M_E = F_{m,v} \times L_b$$

Figure 11.15 - Internal forces of a bolted moment connection

11.7 DESIGN FOR HOLLOW SECTIONS

11.7.1 General design for hollow sections

Clause 11.7 gives recommendations for the design of cold-formed steel hollow sections with nominal thickness up to 22 mm.

Design recommendations: Primary structural members in trusses and portal frames of modest span.

11.7.2 Material Properties

The physical properties of cold-formed hollow steel are given in clause 3.1.6.

The design thickness of the material shall be taken as the nominal thickness.

11.7.3 Mechanical properties

Cold forming is a process whereby the main forming of metal section is done at ambient temperature. It changes the material properties of steel and impairs ductility as well as toughness but enhances strength. The extent to which the properties are changed depends upon the type of steel, the forming temperature and the degree of deformation. Accounting for the changes in material properties, welding requirements as stipulated in clause 11.7.5 shall be followed.

The basic requirements on strength and ductility are given in clause 3.1.2. As a conservative design, no strength enhancement in round corners due to cold-forming is allowed.

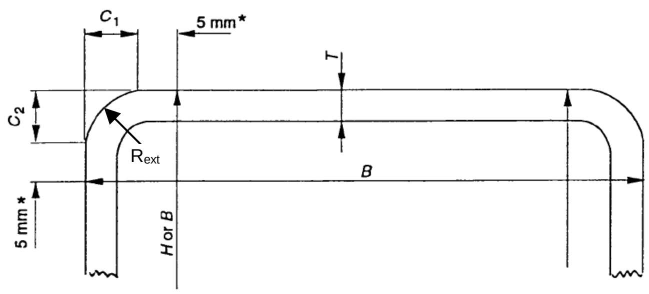
To ensure sufficient notch toughness, the minimum average Charpy V-notch impact test energy at the required design temperature should be in accordance with clause 3.2.

11.7.4 Control of external corner profile

In order to prevent corner bend cracking, control of dimensions of external corner profile should conform to the minimum requirements as stipulated in Table 11.4.

Table 11.4 External corner profile

Thickness t (mm)	External corner profile C_1, C_2 or R_{ext}
$t \leq 6$	1.6 t to 2.4 t
$6 < t \leq 10$	2.0 t to 3.0 t
$10 < t$	2.4 t to 3.6 t
N.B.: The sides need not be tangential to the corner arcs.	



* This dimension is a maximum when measuring B or H and a minimum when measuring T

11.7.5 Welding at cold-formed zones

Welding may be carried out in the corners and the adjacent cold-formed zones, provided that one of the following conditions is satisfied:

(a) the internal radius-to-thickness *r/t* ratio satisfies the relevant value given in Table 11.5; or

Table 11.5 Conditions for welding cold-formed areas and adjacent materials

Minimum internal radius/thickness (r/t) ratio	Strain due to cold forming (%)	Maximum thickness (mm)		
		Generally		Fully killed Aluminium-killed steel (AL ≥ 0.02 %)
		Predominantly static loading	Where fatigue predominates	
≥ 3.0	≤ 14	22	12	22
≥ 2.0	≤ 20	12	10	12
≥ 1.5	≤ 25	8	8	10
≥ 1.0	≤ 33	4	4	6

(b) the welding procedure shall fulfill the Welding Procedure Specification (WPS) as stipulated in clause 14.3.3.

Alternatively, welding may be carried out in the corners and the adjacent cold-formed zones of those cold-formed hollow sections which are produced to those relevant materials specifications of cold-formed hollow sections given in Annex A1.1.

11.7.6 Cold formed section properties under loading

In general, cold-formed hollow sections may be manufactured by forming of the metal at ambient temperature followed by longitudinal weld or spiral weld. The design of bending moment and shear for cold-formed hollow section of this thickness range subjected to various modes of loading may follow the design provisions stipulated in Section 8. For assessing the compressive strength p_c of cold-formed hollow sections, the curve c as stipulated in Table 8.7 of Section 8 may be used.

11.7.7 Calculation of deflection

General

Deflections should be calculated using elastic analysis. Due allowance shall be made for the effects of non-uniform loading.

11.7.8 Connections

Design recommendations on connections and fastenings with the bolt and screw should refer to Section 9.

11.8 DESIGN FOR COLD-FORMED SHEET PILE SECTIONS

11.8.1 General design for sheet pile sections

This section gives recommendations for the design of cold-formed sheet pile sections with nominal thickness up to 16 mm.

Design recommendations: Steel sheet piling is designed as vertical retaining elements in excavation and lateral support works.

11.8.2 Material properties

The physical properties of cold-formed sheet pile sections are given in clause 3.1.6.

The design thickness of the material shall be taken as the nominal thickness.

11.8.3 Mechanical properties

Cold-forming is a process whereby the main forming of metal section is done at ambient temperature. It alters the material properties of steel and impairs ductility as well as toughness but enhances strength. These changes may limit the ability to weld in cold-formed areas. Welding is occasionally carried out at the edges rather than the bent corners of the steel sheet piles.

The requirements of cold-formed non-alloy and alloy sheet pile steel produced from hot rolled strip or sheet with a thickness equal to or greater than 2 mm in respect of chemical composition, mechanical and technological properties and delivery conditions are stipulated in BS EN 10249, BS EN 10149-1 or equivalent standard.

The material specifications of common hot rolled strip or sheet used for cold-forming are:
i) S275JRC / S355J0C stipulated in BS EN 10025-2;
ii) S315MC / S355MC stipulated in BS EN 10149-2; and
iii) S260NC / S315NC / S355NC stipulated in BS EN 10149-3 respectively.

The strip thickness ranges from 1.5 mm to 16 mm for steel, which has a specified minimum yield strength of 260 N/mm^2 up to and including 355 N/mm^2. The available steel grades of alloy quality steels are given in Tables 11.6 and 11.7.

As a conservative design, no strength enhancement is allowed at the cold-formed zones.

To ensure sufficient notch toughness, the minimum average Charpy V-notch impact test energy at the required design temperature should be in accordance with clause 3.2.

11.8.4 Minimum inside radii for cold-formed sheet piles

When cold-formed sheet pile profile is manufactured using JC steel grade, the nominal thickness is limited to 8 mm and the minimum inside radii should conform to Table 11.6 below.

Table 11.6 Minimum inside radii for JC steel grade

Grade designation	Minimum inside radii for nominal thickness (t) in mm		
	$t \leq 4$	$4 < t \leq 6$	$6 < t \leq 8$
S275JRC	1.0 t	1.0 t	1.5 t
S355J0C	1.0 t	1.5 t	1.5 t

Note:
The above minimum inside radii shall apply to JC steel grade only. For tolerances on shape and dimensions, they are specified in BS EN 10249-2 or equivalent standard or equivalent standard. The inside radius to thickness ratio at bent corner of the interlocking crimped end should be limited to 1.5.

When cold-formed sheet pile profile is manufactured using MC or NC steel grade, the nominal thickness can be limited to 16 mm and the minimum internal radii should conform to Table 11.7 below.

Table 11.7 Minimum inside radii for MC / NC steel grades

Grade designation	Minimum inside radii for nominal thickness (t) in mm		
	$t \leq 3$	$3 < t \leq 6$	$6 < t$
S315MC	0.25 t	0.5 t	1.0 t
S355MC	0.25 t	0.5 t	1.0 t
S260NC	0.25 t	0.5 t	1.0 t
S315NC	0.25 t	0.5 t	1.0 t
S355NC	0.25 t	0.5 t	1.0 t

Note:
The above minimum inside radii shall apply to MC/NC steel grades only. For tolerances on shape and dimensions, they are specified in BS EN 10249-2 or equivalent standard. The inside radius to thickness ratio at bent corner of the interlocking crimped end should be limited to 1.0.

11.8.5 Welding at cold-formed zones of cold-formed sheet piles

Welding may be carried out within a length 5t either side of a cold-formed area, provided that one of the following conditions as given in clause 11.7.5 is satisfied.

11.8.6 Cold-formed section properties under loading

In general, cold-formed sheet pile sections may be manufactured by forming of the metal at ambient temperature. The design of bending moment and shear for cold-formed sheet pile section of this thickness range subjected to various modes of loading may follow the design provisions stipulated in this Code.

11.8.7 Calculation of deflection

Deflections should be calculated using elastic analysis. Due allowance shall be made for the effects of non-uniform loading.